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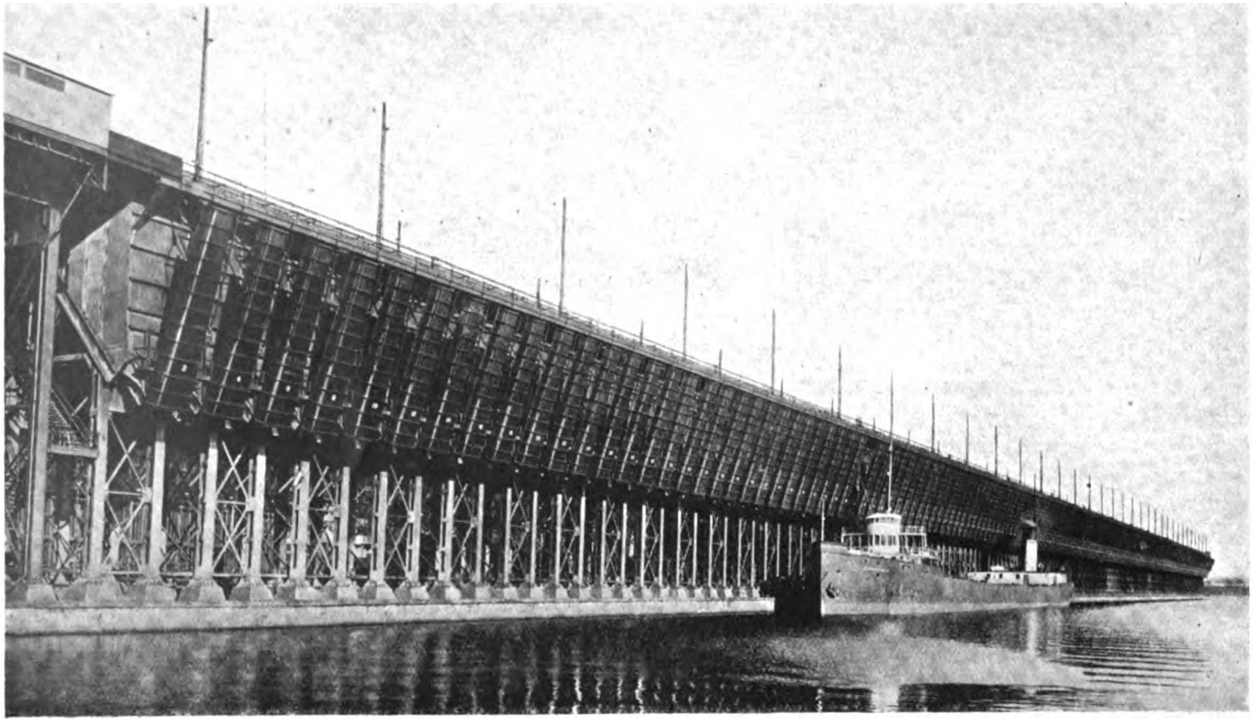

New Ore Dock at Daluth.

(*See p.* 5.)

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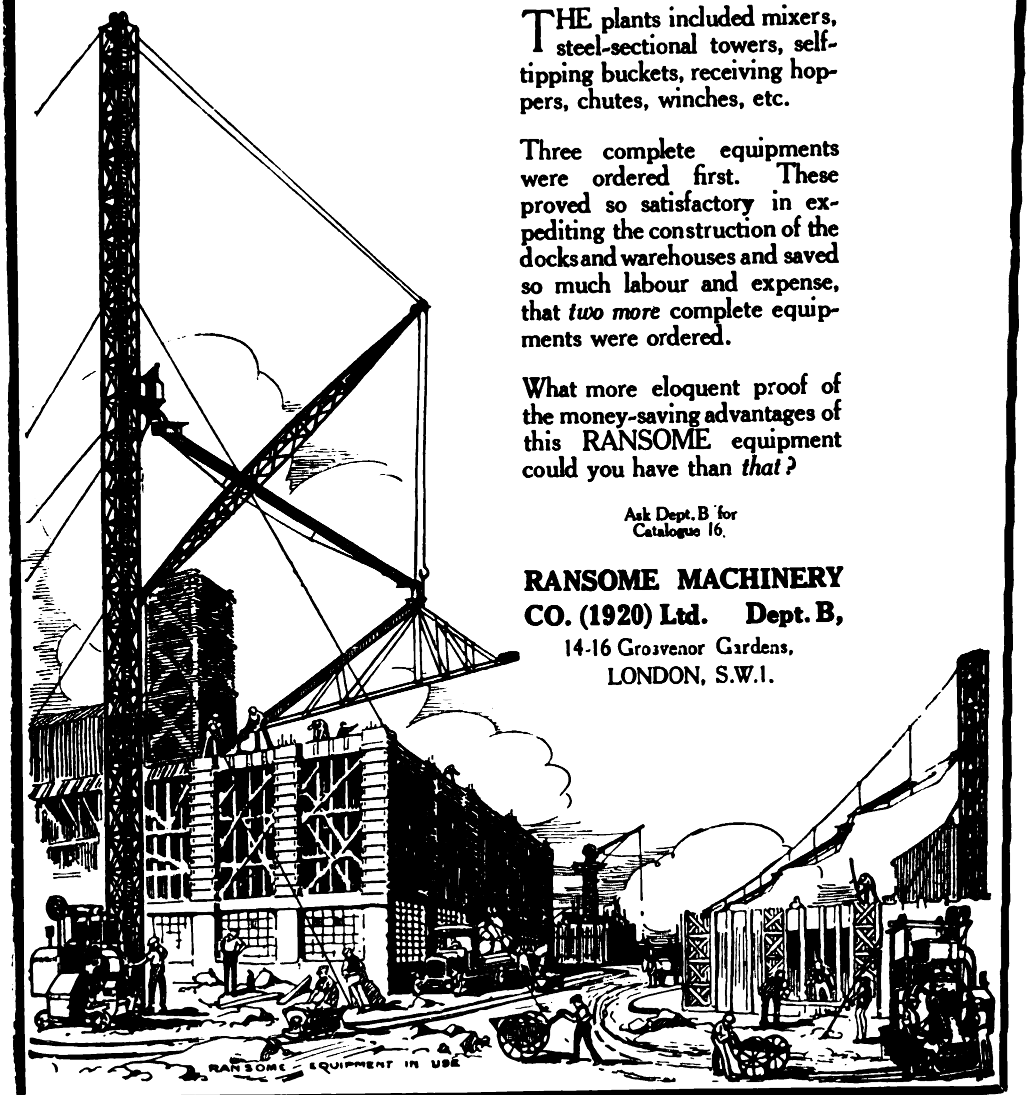

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XVIII. No. 1. LONDON, JANUARY, 1923.

EDITORIAL NOTES.

THE BUILDING OUTLOOK: AND SOME SUGGESTIONS.

FOLLOWING the long period of depression through which the building industry has been passing, there are signs that the time is not far distant when the industry will at any rate set its feet on the road to more prosperous times than has been its lot during recent years. For a long time the public has hesitated to spend money on building—or for that matter on any other commodity involving any considerable sum of money—in the hope that prices would fall, and in many cases that policy has been justified. There is now a feeling, however, that if the cost of building has not actually reached a level which will be stable for many years to come there will not be further reductions of any magnitude for at least another year or two, and the trend of prices during the past few months goes to show that this will prove to be correct. It certainly seems doubtful if prices of materials will be reduced by any considerable percentage in the near future—some materials, indeed, show a tendency to increase—so that if any appreciable decrease is to take place it can only be by wages reductions. Here it is possible that some slight saving will be shown in the next month or two, for it has to be remembered that the wages agreement between employers and employed in the building industry has placed the building operative in an enviable position compared with workers in other industries, and the present unemployment will no doubt be in the minds of the men's leaders when the question of wages comes up for reconsideration in the near future. But just as it was impossible to be certain when the peak had been reached when prices were soaring, so it is very difficult to say exactly when the lowest point has been reached on a falling market. We do not say that building costs will not go lower still before the post-war normal is reached, but it does not seem probable that there will be any general lowering of the cost of building in the near future sufficient to influence those who are in need of a building, whether for business or for private use. This is no doubt realised by the general public, and is the cause of the increased number of inquiries now being received by the industry.

Unfortunately, the prospects of a revival have come at a time when from force of habit, and without due consideration of the difference in present-day building methods compared with those of a generation ago, a large proportion of the general public—including many prospective building owners—cling to the idea that practically all building work has stopped on account of the weather. The fact that with up-to-date methods buildings can be erected all the year round is not sufficiently realised, and it is not realised because the public is not told of the progress which has been made in the science of building during recent years. Some industries go to a great deal of trouble and expense to keep the public informed as to their possibilities and developments, but the building industry seems to be singularly lax in this direction. The gas industry, for example, has done much to increase the output of the gas producers and manufacturers of gas fires and stoves by making known the advantages of gas heating and cooking; by decreasing the price of power for domestic use during the slack period in the middle of the day the electricity undertakings are not only helping the manufacturers of electrical apparatus, but are also placing their undertakings on a more economic basis by keeping a larger proportion of their plant in operation at times other than the morning and evening periods of maximum demand; the London tramways and omnibuses are carrying passengers at a very low fare in the middle of the day in order to attract more passengers and thus get some return at times when otherwise the vehicles would be running almost empty. To get nearer home, we may instance a branch of the building industry—the painting trade—in which the combined efforts of the paint manufacturers and the decorating firms last year did much to ease the usual rush for decorating work at Easter by advising the public to put such work in hand earlier in the spring and, we believe, lower prices were quoted for work done before the rush period.

By spreading work over a longer period, whether the slack period to be eliminated be a few hours in the day or whether it extend over several months, overhead charges are reduced, and labour is more stabilised, two important considerations from the point of view of the builder, but it is doubtful if anything in this direction will be achieved without a publicity campaign. In America propaganda to make building an all-the-year-round industry has been going on for some years: "There is no excuse for anything less than a twelve months' construction season—except the excuse of bad habit" we read from a recent pamphlet on the subject, and we heartily agree. No one denies that winter building is more expensive than summer building, but the essential point is to dispel the idea that it is impossible, for there must be many instances of large buildings which are urgently required being postponed until the spring simply because of an idea that they cannot be commenced until the cold weather has passed, and not on the score of any possible extra expense. And providing proper precautions are taken to prevent frost getting at the work (and these protective measures are now well known) buildings erected during the winter are every bit as sound as those built in warmer weather.

Could not something be done to make the possibilities of winter construction more generally known in this country? It is worth considering whether money spent in popularising winter building would not be amply returned if building became less of a seasonal and more of an all-the-year-round industry, for not only would the overhead charges of the building firms be reduced by their organisations being kept intact, by plant being continually in use instead of lying idle for several months each year, and by being in a better position to keep their operatives together, but the nation would benefit by the reduction of the unemployment which is now, and unfortunately always has been, so prevalent in the building and allied industries during the worst months of the year. Certainly building is different from gas, electricity, and tramway undertakings, but bearing in mind the success which has attended the efforts of these services to increase their off-season revenue by granting concessions to their customers it is even worth a thought as to whether it would not pay in the long run if builders accepted a smaller margin of profit for winter work in order to reduce the extra cost to the client. We know on many buildings at present going up the margin of profit to the builder is almost non-existent, but these are exceptional times which will not always be with us. At any rate our suggestion might go a long way towards getting building spread over a longer period during the year. What we want is for the public to get the "winter building habit," but that can hardly be brought about without an organised publicity campaign throughout the country to let the public know what can be done in the constructional field during the colder months.

COLOUR IN SMALL HOUSES.

ONE does not need to make a deep study of sociology to realise the effect of environment on character, for who can walk through the slum areas of our large cities without sympathising with those whose lot it is to dwell in them? And who in a more fortunate position, if he but give it a thought, does not feel that living in such a neighbourhood his outlook would in time become as hopeless as is that of those who are condemned to live in an atmosphere of squalor and dreariness which no effort of their own can appreciably improve, a drabness which nothing short of complete rebuilding can ever remedy? Slums have been allowed to grow to such an extent that nothing but Government measures on a huge scale can effectively solve the problem, but much can be done to prevent the growth of the evil by careful attention to the new houses which are being built and by making every effort to prevent them from becoming the slums of to-morrow.

In looking at the dwellings which make up the slums of to-day, does any one wonder that they are slums? Crowded one upon the other in endless rows, often with the front door opening direct from the living room on to the pavement, with windows overlooking across tiny paved yards at the back (and often at the front too), with no evidence of an atom of thought having been given to the interior arrangements with a view to the convenience of the occupants—could they be anything else than slums? Now and again one hears of efforts being made to "brighten" slum districts—such as the recent experiment in Bermondsey, where the Borough Council endeavoured, and with a certain measure of success, to get the inhabitants to grow flowers in window boxes. But the responsibility for the ugliness of the slums rests not with those who live in them so much as with those who built them and those who allowed them to be built. In such a setting as a slum, what incentive is there to make it attractive? It is too much like trying to be cheerful in a cemetery.

If people are to take a pride in their homes and their towns the first essential is to give them something of which they can be proud. No amount of persuasion will make a man or woman take an interest in a home which is inherently ugly outside and inconvenient within, but the appearance of the new Government houses shows that the same people who neglected the miserable slum dwellings from which some of them were transferred vie with one another in keeping their new homes neat and tidy and clean. No one denies that one of the first causes of the present unrest among the workers, the most fertile soil for the propagation of Bolshevist doctrines, is the condition under which the bulk of the working classes are housed, and the only solution lies in the sweeping away of the slum areas and rehousing the people in decent homes. But as nothing on a big scale seems possible in the near future, at any rate no effort should be spared to make the new houses something in which the tenants will be proud to live. On the whole, and bearing in mind the difficult times in which they were built, and the cries for economy which led everything to be cut down to a bare minimum, the houses already built under the Government scheme will go a long way to foster that " home-pride " which is so necessary to good citizenship, and they are certainly a vast improvement on the general run of working-class houses built before the war. If a new Government scheme is to be put in operation shortly, as seems probable, and if, as has been hinted in Parliament, the responsibility falls on private builders as well as municipal authorities, it is to be hoped that a close control will be kept to see that the standard set up for this class of dwelling in the past few years will not be lowered, and that if possible it be improved upon.

It is with the object of showing the possibilities of making small houses more attractive by the judicious use of colour that we give this month some illustrations of the coloured concrete houses built by the Liverpool Corporation under the direction of Mr. F. E. G. Badger, A.M.Inst.C.E., the Director of Housing, who are to be congratulated on their enterprise in showing what can be done in this direction. The use of colour in working-class houses should go a long way towards preventing them from becoming as drab and uninteresting as the great bulk of such houses with monotone exteriors have become, and the use of concrete blocks is probably the cheapest, and certainly the most durable, material for the purpose. An account of the coloured concrete houses illustrated is given on p. 23.

AMERICAN STANDARD SPECIFICATION FOR PORTLAND CEMENT.

A TENTATIVE specification for compressive strength of Portland cement mortars has just been issued by the American Society for Testing Materials for the purpose of eliciting criticism and suggestions ; when finally approved it will be included as part of the American Standard Specification for Portland Cement. It is proposed that the minimum compressive strengths of a mixture of 3 parts of standard sand and 1 part of cement should be 1,200 lb. at 7 days and 2,000 lb. at 28 days. These are surprisingly low requirements, and will do little towards raising the standard of quality of cement. The recommended form of test piece is a cylinder 2 in. in diameter and 4 in. in length, and this small size should allow of a simple and inexpensive testing machine being devised which will overcome one of the present objections to compression tests, namely, the high cost of the testing machine. A useful condition of the specification that might well be applied to tensile tests is that individual tests differing more than 15 per cent. from the average value of all test pieces, tested at the same period and made from the same sample, shall not be considered in determining the compressive strength.

A LARGE ORE-SHIPPING DOCK.

By W. H. HOYT, M.Am.S.C.E.

THE new ore dock of the Duluth, Missabe & Northern Railway Company, on Duluth-Superior Harbour, completed recently, is the largest ore-shipping dock in the world. It has a length of 2,438 ft., and is big enough to dock two of the world's largest ships (the *Bismarck* and the *Leviathan*) on either side, lying entirely inside the outer end. The deck is 84 ft. above water-level, with electric light masts reaching 120 ft. above water-level.

FOUNDATIONS.

Four drop-hammer machines with 60 ft. leads were used on the pile-driving work, and a total of 971,014 lin. ft. (or over 183 miles) of timber piling was driven in seven months. During this time two other machines drove 174,230 lin ft. of 12-in. United States steel sheet piling, 43 lb. per ft., in 38-ft. lengths. The timber foundation piles are from 50 to 85 ft. in length, and were designed to be loaded up to 20 tons per pile maximum. The pile-drivers were swing machines carried on platforms on piling driven ahead, followed by the steel sheet pile-drivers and then by the outside fender-driving machines. The piles were driven to 9 in. above low-water datum (about average water-level), and 180,000 ft. B.M. fir timber was used for caps. Cross-tie rods, 1¼ in. in diameter, every 12 ft. across the dock anchor the tops of the sheet piling in place. The entire dock foundation when surrounded by steel sheet piling was pumped full of sand, 33,000 cu. yds., from slips alongside and 49,000 cu. yds. brought in by scows. The sand was carefully levelled off at low-water datum and served as a support for the concrete foundation slab, leaving the piles projecting 9 in. in the concrete.

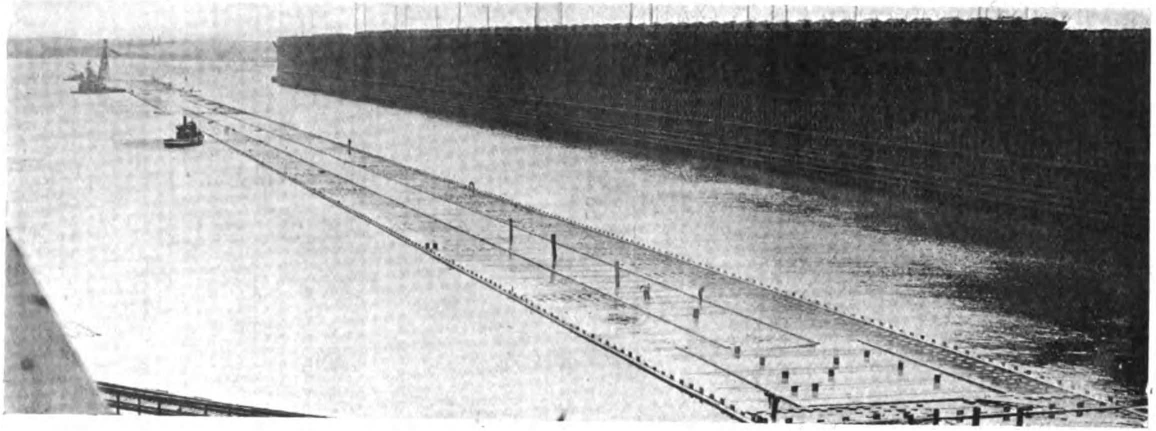
The foundation of the dock is a reinforced concrete slab 2,438 ft. long, 69 ft. wide, and 6 ft. deep, with openings along the centre line for filling with sand. A 1 : 3 : 5 mixture was used of Portland cement, Lake Superior sand, and crushed trap-rock crusher run, taking everything through a 3½-in. screen. The reinforcement consisted of a layer of 1-in.-diameter plain round steel rods (6-in. centres) crosswise of the dock, and a layer of ¾-in. diameter rods (12-in. centres) lengthwise of the dock, placed near the bottom of the slab. A total of 884,410 lb. of reinforcing steel and 171,320 lb. of the rods was used in this part of the work. Recesses were left in this concrete slab to allow the placing of the column anchor bolts and the pouring of the column footing pedestals as one block. This avoided the horizontal joint line at the top of the slab and permitted simple handling of anchor bolts. In this work 28,698 cu. yds. of concrete was placed in four months. Expansion joints are provided at cross sections 144 ft. apart in the foundation slab as well as in the superstructure. An oak wale strip 12 in. by 16 in. is embedded in the face wall of the foundation, set into the concrete 8 in. and held in place by anchor bolts every 6 ft.

A 1¼- in. water-pipe was laid the whole length of the foundation of the dock to supply drinking water to bubbling fountains on the dock. This pipe was laid 3 ft. 9 in. below the top along the centre and covered with sand, thus ensuring a cool supply during the summer. The pipe was laid with a grade and drain outlets in order that it might be drained to prevent freezing in winter when the dock is not in use.

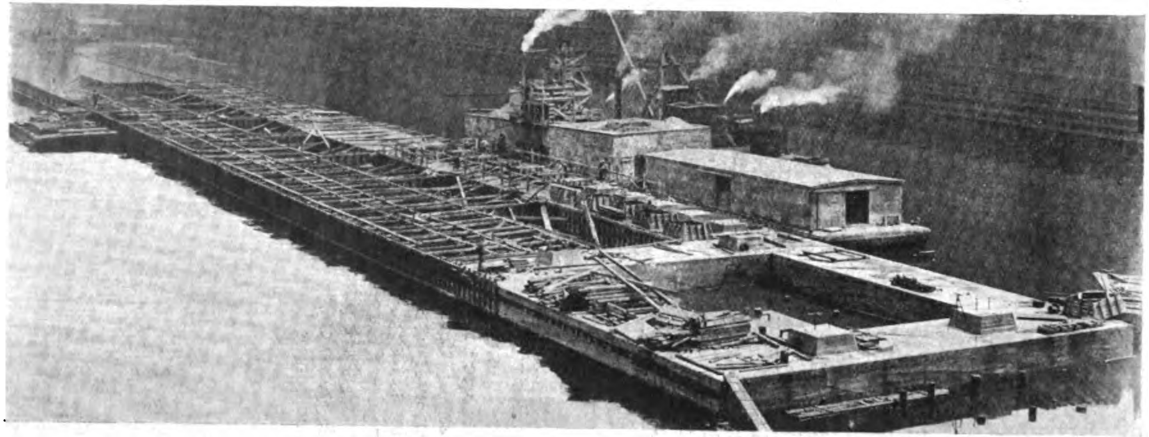
SUPERSTRUCTURE.

The superstructure provides for 384 ore pockets (12-ft. centres) with 6,540 cub. ft. capacity each. Eight standard 50-ton ore cars can be dumped into each pocket without trimming, thus providing a storage capacity for 3,072 cars or a total of 153,600 tons. The top of the dock is 76 ft. 5 in. wide, and carries four tracks along its entire length. Seven lines of walks furnish ample working room and safety for employees. The elevation of rails and walks was set to leave no flange-way openings to require cleaning. The entire dock and approach are designed to carry the heaviest locomotives of the Duluth, Missabe & Northern Railway, the 304-ton " Mallet " type engines and the 266-ton " Santa Fe " type, with 55,000-lb. axle loads.

The superstructure is of structural steel frame with concrete partition walls, pocket walls, and sidewalk slabs. The fronts are of plate steel reinforced with heavy I-beams and tie bolts. The whole is supported by two lines of steel I-beam columns spaced 12 ft. apart, each column



Foundations, Showing Sheet Piling.



Foundations, Showing Piers.

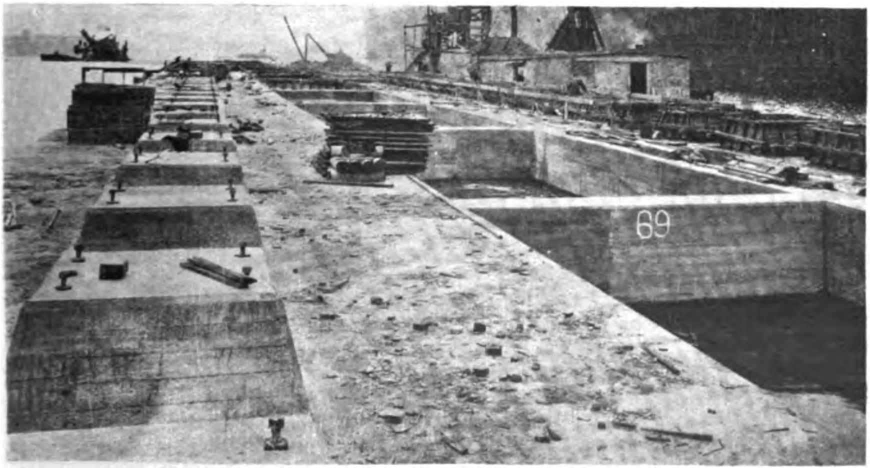
NEW ORE DOCK AT DALUTH.

designed to carry 1,500,000 lb. Three 24-in. I-beams, 115 lb. sections, stayed with batten plates, are used for each column. The partition walls and loads from the pockets are carried to the columns by sloping plate girders, and the pocket bottom slab rests on heavy I-beams connected at the ends to the main girders. Reinforcement is put into the floor slab, both for slab action across the supporting beams and for shear between the slab and the side walls to prevent the slab from sliding out of the pocket. The whole structure has wind and sway bracing to prevent distortion. Special care was taken to provide effective expansion joints every 144 ft. of the length of the dock.

The concrete for the superstructure was made of carefully-graded material in proportions by volume, with one part cement, three parts sand, five parts crushed stone, and one-quarter part of hydrated lime. The hydrated lime was added to secure easy running concrete around and under reinforcing bars, and to assist in waterproofing. The surface of the concrete after removing the forms was very even and required a minimum amount of hand-pointing work. Plain round bars suitably bent were used to reinforce the bottoms of the ore pockets. The top deck was reinforced with suitable sizes of American Steel and Wire Company's "Triangle Mesh" reinforcement.

Concrete bumping blocks, bonded into the steel and concrete floor, finish the outer ends of each track to prevent cars being pushed off the end of the dock into the Bay. All stairways are of steel and concrete, and underneath the ore pockets a 4-ft. concrete walk, elevated above the foundation slab, was placed the full length of the dock to provide a dry walk above the sand and water when, during the pumping out of ballast water from the ore boats, this deck is flooded. The ore chutes are of the usual type but heavier, and are raised and lowered by specially designed hoists.

The dock has a two-track approach of concrete and steel 3,087 ft. long. To reach this section an elevated section 400 ft. long was built of two concentric reinforced concrete retaining walls 24 ft. apart and 15 ft. to 8 ft. high, held together by reinforcing rods and cross walls spaced 20 ft. apart. This was filled and the track laid on top on rock ballast. The long steel approach has plumb steel columns resting on concrete pedestals and piles, with standard plate girders and concrete trough deck of pre-cast reinforced slabs. The longest girder span is 80 ft. 6 in., and 30-ft. spans are used for tower bents. A portion of the approach where the track area is required to be wider to meet the four tracks on the dock was built *in situ* by using forms and placing reinforcing steel to give the wider construction but continuing the same curb design. All electric wiring is enclosed in



New Ore Dock at Duluth: Showing Foundations.

New Ore Dock at Daluth: Steelwork during Erection.

conduits on the outside of the concrete slabs.

Some of the larger quantities of materials used in the construction of this deck are as follows: 1,067,971 lin. ft. of timber piling (50 ft. to 85 ft. in length); 174,230 lin. ft. of 12-in. 43 lb. U.S. steel sheet piling; 162,570 sq. ft. of "Triangle Mesh" reinforcement; 59,846 cu. yds. of concrete in the dock and the approach; and 29,609 tons of steel.

The dock was designed and erected by the following personnel of the Duluth, Missabe & Northern Railway Company: Messrs. W. H. Hoyt (Assistant Chief Engineer), W. E. Hawley (Office Engineer), A. M. Frazee (Electrical Engineer), F. C. Baluss (Engineer of Bridges and Buildings), and F. N. Graham (Engineer of Construction).

The total cost of the dock is over 3,500,000 dollars.

Concrete at the British Empire Exhibition.

In an article in a contemporary on the construction of the buildings at the British Empire Exhibition at Wembley Park, Mr. E. O. Williams, Consulting Engineer to the Exhibition, says: "Exterior walls (of the Stadium) of no structural value are of cement-faced breeze blocks resembling masonry. The columns supporting the roof over the stands are slender octagonal, pre-cast reinforced concrete. By using reinforced concrete in preference to steelwork for exposed surfaces, maintenance charges are reduced to a minimum and the beauty of the structure enhanced. Structural steelwork was used for interior framework because of (*a*) present low cost of structural steel; (*b*) speed of construction; (*c*) the greater part of the framework is at a considerable height, over filled ground on which it would not be safe to build the falsework necessary for reinforced concrete beams. The octagonal reinforced concrete columns were adopted as, contrary to original expectations, they permitted the minimum dimensions with least obstruction to the view of the spectators."

DESTRUCTIVE MARINE BORERS.

By GOWER PIMM, A.M.Inst.C.E.

Of all the problems which present themselves to those responsible for the construction and maintenance of harbours and marine works generally there is probably none with which it is more difficult to deal than the ravages of destructive borers. Other destructive agencies—powerful as they are—can at least be foreseen with more or less certainty, and provided against by suitable design in most of the usual materials of construction, but it is in their ability to resist destruction by animal agency that the limitations of many materials, otherwise suitable, become most apparent. In this respect there can be no doubt that concrete and reinforced concrete are supreme.

The marine borers with which the engineer is concerned may be classified as:—

(1) Crustaceans which attack timber.
(2) Molluscs which attack timber.
(3) Molluscs which attack rock.

These three classes include creatures ranging from the microscopic to the gigantic, and it is difficult to say which are the more destructive.

Of the first kind—timber-boring crustaceans—the best known is the *Limnoria Lignorum* (or Gribble), which though scarcely larger than a flea is capable of causing the most extensive damage to timber in marine works. It carries on its work of destruction progressively from the surface inwards, and the outer portions, being quickly reduced to a spongy condition, crumble away behind the borer, which continues its ravages until the whole of the timber may be destroyed.

The *Limnoria* sometimes works alone and sometimes in company with a similar but somewhat larger crustacean, the *Chelura Terebrans*, the latter, however, being seldom found without the former.

These pests do not like creosote, but timber treated with this substance is by no means immune from attack, as they merely vary their operations by first penetrating to the interior through any shake or defect they can find and then work outwards until they reach the impregnated surface, leaving the timber apparently sound but in reality nothing but a shell. These borers, in common with others, naturally prefer the softer kinds of timber, and the softer parts of any particular kind. For example, they will leave the knots of a soft-wood untouched, but, on the other hand, they will attack the hardest varieties. They have been found in nearly every kind of timber used in constructional work, including Memel, pitch pine, greenheart, Canadian rock elm, oak, and Tasmanian blue gum, and have caused enormous damage in most British harbours.

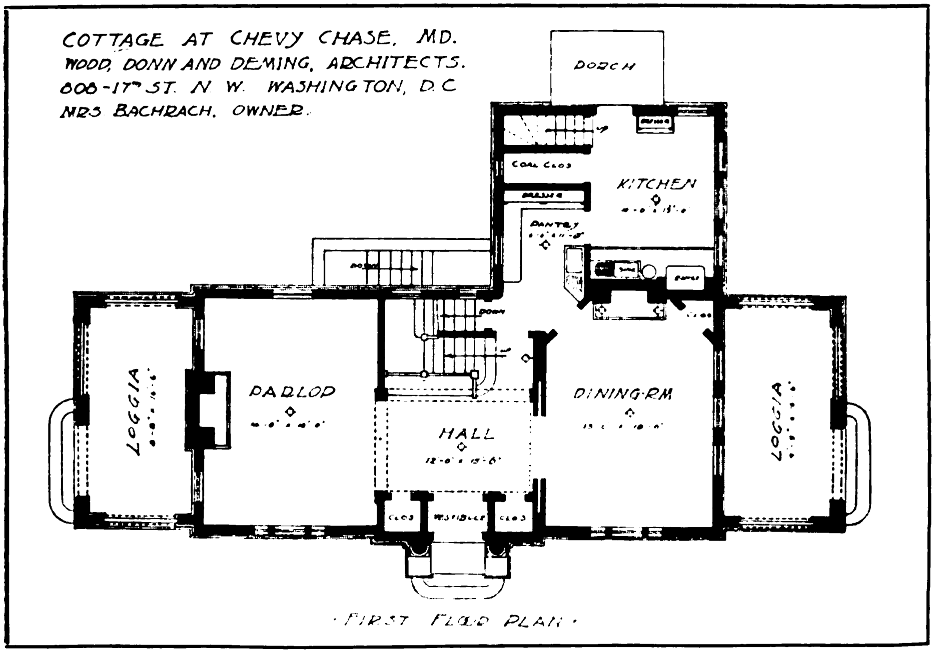
The second kind of timber-borers—the molluscs—while not quite so ubiquitous as the *Limnoria*, are nevertheless found in many British harbours. The best known are the *Teredo Navalis* and the *Pholas*. The former, when quite young, attaches itself to the surface of the timber and bores a tiny hole through which it enters, and this hole, which is a mere pin-hole, is all that is seen upon the surface, but the ravages of the creature in the interior are terrible. Its head is formed of two "valves" with sharp cutting edges by means of which it bores a perfectly cylindrical hole along the grain of the timber, and as the *Teredo* grows and advances the hole of course increases in size—up to half an inch or more in diameter and many feet in length. The hole is lined throughout with a hard, shell-like substance. In a badly-attacked specimen the holes are separated by partitions sometimes no thicker than a sheet of paper, but in an average specimen they are usually about their own diameter apart. The *Teredo* is found in pitch pine, Memel, Canadian rock elm, Jarrah, greenheart, and probably most other kinds of timber.

The other timber-boring mollusc—the *Pholas*—is not unlike the ordinary mussel, and while larger than the *Teredo*, in this country at all events, its ravages are not quite so serious as it has not the same facility for eating its way through the timber.

Many methods of protecting timber from the attacks of crustaceans and molluscs have been tried, such as sheathing the timber, treating with creosote, or with molasses burnt in with a blow-lamp, but all of these are necessarily somewhat superficial, and possess the disadvantage that the timber must first be fitted and all necessary bolt-holes bored and afterwards treated, as otherwise the newly-exposed untreated surfaces provide a starting place for the pests.

Some species of *Pholas*, and other bivalves, are also found in rock, and have done considerable damage in several British ports. They attack principally limestones and the softer sandstones, but the writer has not heard of any case in which they have attacked the more compact rocks. The only instances of these molluscs being known to attack concrete are believed to be confined to works in which the aggregate has been one of the softer kinds. It would therefore appear that the use of concrete made from a suitable hard aggregate is the best possible means of ensuring immunity from the ravages of destructive borers of all kinds.

These notes would be very imperfect without some reference to the work of the Committee of the Institution of Civil Engineers which is investigating the whole subject of the deterioration of structures exposed to sea-action, and the writer desires to express his indebtedness to the Secretary of the Committee for permission to make use of information obtained at the Museum formed by the Committee as to the prevalence of the various destructive borers. In this connection one point of particular interest may be mentioned, as it illustrates the advantage of organised research over the results of individual observation. In dismantling a timber structure in a British harbour some years ago it was found that the pitch pine rubbing pieces were riddled by *Teredo* but that the jarrah fenders were in perfect condition. As both were fixed at the same time, viz., about seven years previously, the inference appeared to be that *Teredo* could not attack jarrah. Yet amongst the exhibits referred to above is a specimen of jarrah, also from British waters, which provides as good an example as could be found of the ravages of the *Teredo*. It would be interesting to know whether in the case mentioned the jarrah owed its immunity from attack to the presence of the pitch pine, and perhaps the Committee, in some future report, will throw some light upon this point.



[Messrs. *Wood, Donn & Deming, Architects.*

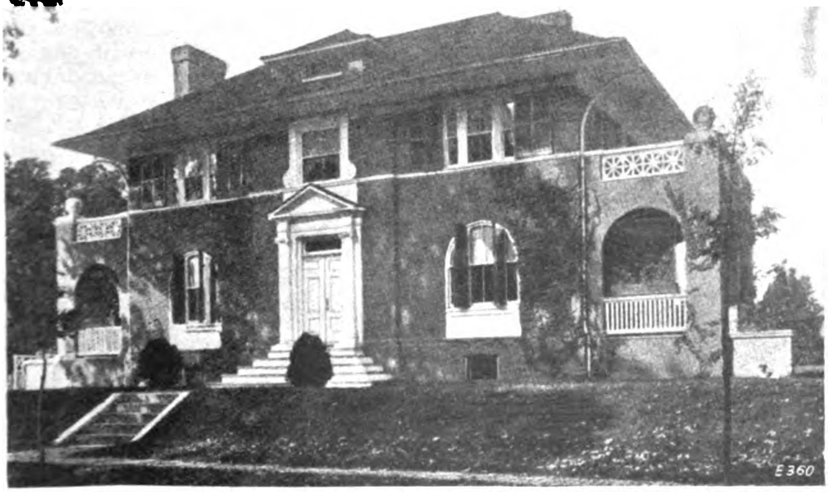
House at Washington, U.S.A. (*See p.* 11.)

AMERICAN DOMESTIC ARCHITECTURE.

In the course of our survey of American domestic architecture executed in concrete we must not confine ourselves to the study of the elevations, but where the material is available extend our examination to the plan, for by so doing, and climatic differences between England and America notwithstanding, we shall find matter of value and of interest. The house illustrated (cottage is the heading inscribed on the plan, but our view of cottage architecture would scarcely include anything quite so spacious) shows a plan composed of a few large rooms providing the accommodation for which there is just now a great demand. The whole is arranged on broad symmetrical lines, the two large sitting-rooms being placed abreast a generously-dimensioned hall. The kitchen and pantry arrangements might well be studied in detail, for they suggest a great saving of labour as compared with our often more complicated and wasteful devices, although our custom of placing the sink under a window has much to commend it both from the point of view of construction and convenience.

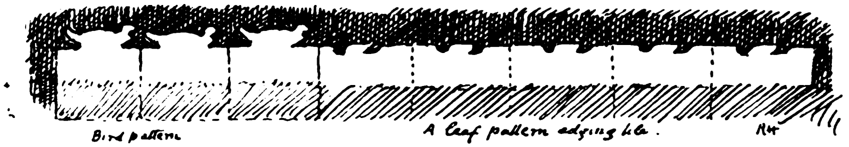
The composition has its interest focussed on the door and the superimposed window, with subsidiary masses formed by the logias, and slight emphasis given by the projection of the central feature. That it is the work of a skilful designer is patent in many details, such as the one central dormer window completing the central mass, in the horizontal divisions made by the string-course, which is made to align with the coping of the adjoining parapet, thus binding together the whole composition, and which forms for the first story a kind of frieze.

The house is constructed in concrete, and the system employed is that of concrete on expanded metal lathing. It will be noticed that the surface appears quite dark in contrast with the white paint-work, showing that the American architect is not at a loss for methods of surface treatment.



[*Messrs. Wood, Donn & Deming, Architects.*

House at Washington, U.S.A.


Bird pattern
A leaf pattern edging tile.
RH

Concrete Edging Tiles.

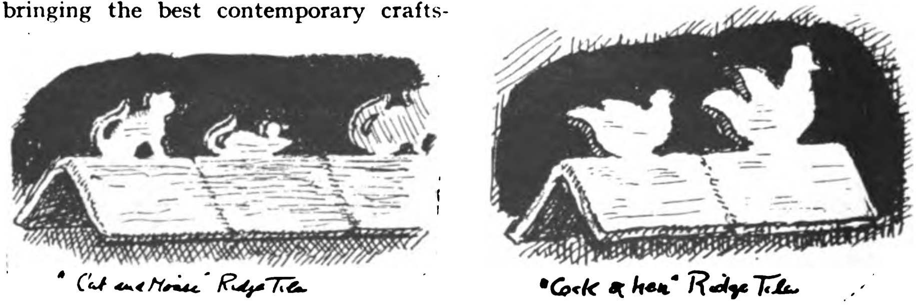
ORNAMENT IN CONCRETE.

BY REGINALD HALLWARD.

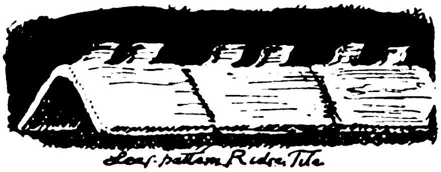
A good deal of the criticism brought against industry, that it is either indifferent to art or only interested to make use of it for commercial profit, so abusing its nature, is very much beside the mark. We can hardly be said to abuse that of which we do not know the nature, and we must look much deeper into the reasons for the separation of art from industry if we are to understand and then remedy what is lacking in the relationship between commerce and art. The work of restoring the natural corporate relation between art and industry will no doubt be a long process, but it has more than ordinary opportunity for manifesting itself through a material like concrete, which is so young in its modern use and to which the traditions which burden other materials do not cling. It is for this reason that in considering the treatment of ornament in concrete we should avoid looking at it through the traditions and precedents that govern other materials, but make our design frankly the expression of the material and of our own time, accepting its limitations and working through them. Unfortunately, from the great amount of merely imitative or derivative design, we have come to regard ornament as something comparatively easy. Successful results will not follow in copying examples of Renaissance or other ornament, but through bringing the best contemporary craftsmanship to exercise itself on the material, by showing what the modern designer and craftsman can create through the material itself.

That concrete lends itself to ornament in a variety of ways, in relief or incised, for friezes, panels, stringcourses, mouldings, in all the many directions which architecture includes is not to be doubted. In certain ways incised or sunk ornament seems of special adaptability to the material as leaving it less exposed, and I have seen examples of this treatment which offered a pleasing variation to the moulded portions; the more direct hand craft of the ornament contrasting well with the latter.

Lettering, which is such an important feature and applicable to so many purposes, for inscriptions, signs, etc., can either be incised or cast in relief. Original ornament in relief or incised can be cast if reproduction is desired, and afterwards worked on with tools to restore any loss of sharpness and treatment. A method of executing ornament in concrete, such as a panel in relief, is as follows: The design having been planned—for which a cartoon will have been made—a mould is made from it, blocking in the masses and establishing the principal reliefs, and while still soft the blocked-in masses are carved on, to the completion required, while the material is still soft, slow-setting cement giving


"Cat and Mouse" Ridge Tile
"Cock & hen" Ridge Tile

Concrete Ridge Tiles.



Leaf-pattern Ridge Tile

quite sufficient time for this to be done.

There has not been at present much development in the way of producing the work of modern designers and craftsmen, and one would like to see some examples of contemporary work in concrete. These no doubt will come, and when the modern artist is given opportunity to work in the material we ought to see some striking and interesting results. It is not enough to cast and carve from old models ; we want the fresh inspiration of our own time, native to the material and evolved from it, not imitations of Classic, Renaissance, or other design.

A field for the development of the use of concrete for ornament exists in the direction of headstones and other memorials, and as the vagaries of personal choice cause a great deal of disfigurement to our cemeteries it would be interesting to see some designs carried out in concrete for a headstone of a good standard type. If a headstone, of becoming simplicity and distinction of character were available, there ought to be a considerable opportunity for its use, and at less cost than carved stone or marble.

There is a great advantage in the colour of the mixtures of concrete now available, and the varieties of surface. In whatever way we are using the material its use should be governed by its own nature, and we should avoid the imitations of other materials which custom and habit and a want of mental energy tend to impose on us. Let me instance this defect in the case of the small edging tile. The garden edging tile is in most cases rather an ugly thing, and therefore disfiguring to the border and to the path. To any one with appreciation of what is attractive for garden walks, most of the ordinary edging tiles will be ruled out. The " Cock and Hen " ridge tile is the memory of some tiles of a yellow colour, I came across on an old rectory in Essex, probably modelled in a local clay, and with all the freedom and fancy of a craftsmanship free enough to find pleasure in it. The " cat and mouse " ridge tile follows naturally on the former, and was sketched for the same reason.

The edge of a garden border is not in the first instance a suitable place for ornament of any detailed kind, which will not be properly seen and is wasted in such a situation. Anything like a ragged edge is particularly to be avoided and it should be governed by horizontal lines, not vertical. The ridge tile offers another opportunity for the designer in concrete ; the modern ridge tile is seldom satisfactory, though there are old examples, such as the seventeenth-century cresting on the roof of an old vicarage in Essex, to which I have referred, which are admirable. The late Victorian period was responsible for an excess of unsuitable and ugly ornament of the kind we see on so many houses of that period, when all significance had gone out of it. The present time no doubt is hardly one to indulge in unnecessary ornament, and unless it is really the result of a genuine impulse to expression, to give form to beauty, we are better without it. Walk out of London in any direction and the houses we shall like best, the architecture which will most appeal to us, will be that most free from ornament and ostentatious details. Concrete fortunately carries no such load of tradition as to its use that it need fall into this error, and we can be well content when using ornament to make it the vital expression of our own time and not that only of our forbears. We can learn something from the past in considering the character and requirement which should govern the use of ornament, and we shall find that Gothic ornament, for instance, develops on a certain standardisation of types. Variety there is, but it is a variety in unity, an underlying common impulse towards a similar end. I can perhaps make this plainer by an illustration, and as a homely instance we may take the edging or ridge-tile. If we could create one of fine and fundamental character in ornament, it would be much better to stick to this very jealously rather than to be for ever craving for novelty and new shapes. When we, as in the Gothic period, put a first-hand conviction and inspiration into our work we shall reject the multiplication of second-rate designs and novelties, from a discipline more identical with the Gothic which establishes a tradition out of a few fine types.

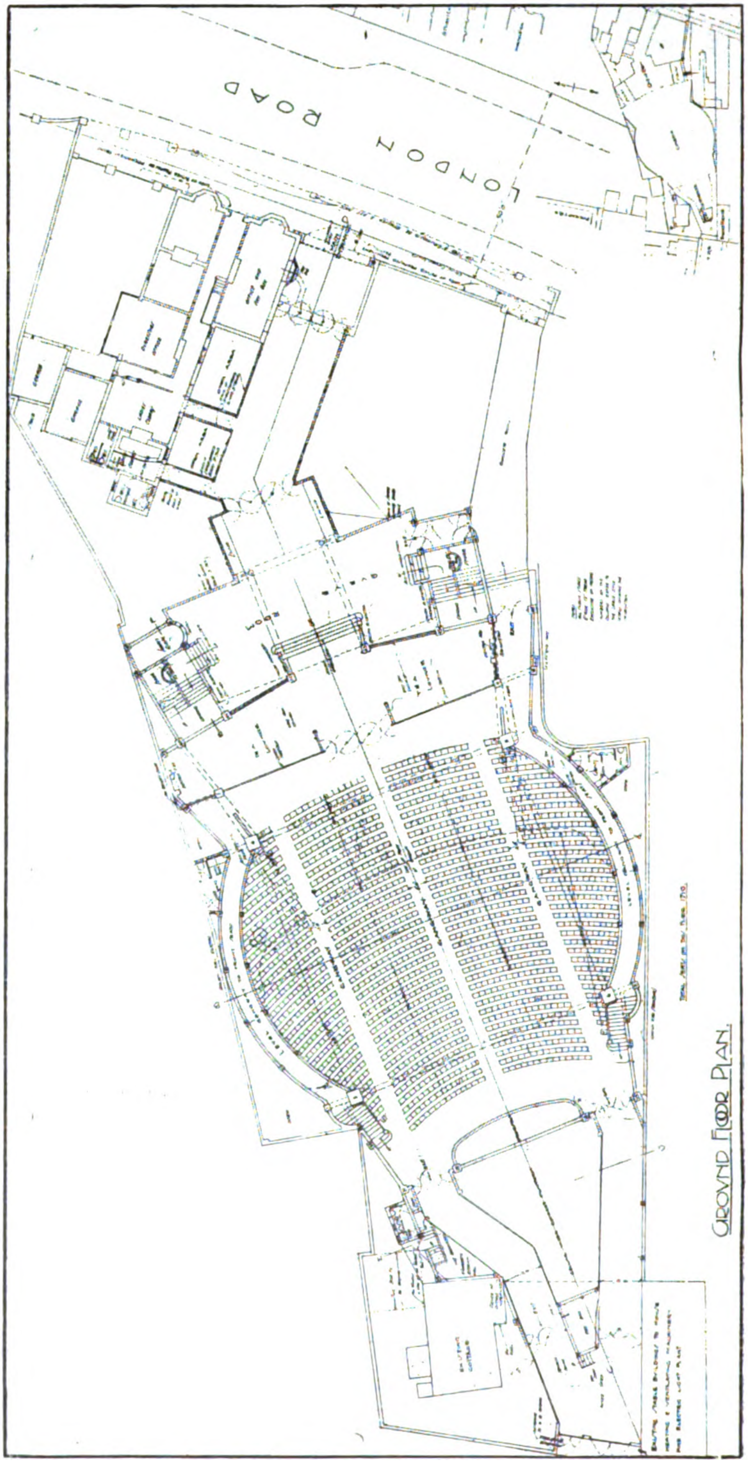


Fig. 1.—Reinforced Concrete Cinema at Portsmouth. (*See p.* 15.)

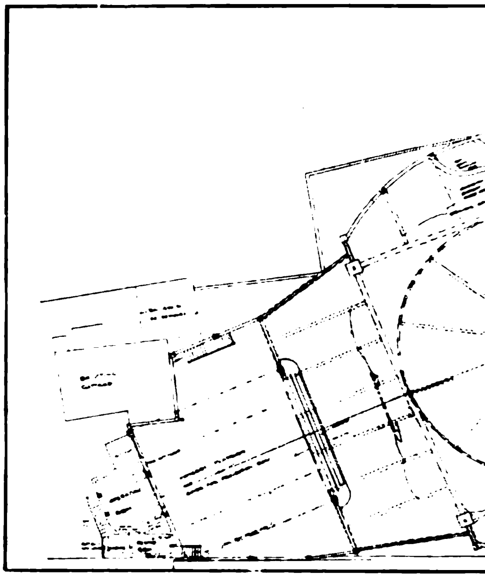
REINFORCED CONCRETE CINEMA AT PORTSMOUTH.

IN an Editorial in our October number the constructional side of theatres was treated, with a view to laying before the architect and constructor the exceptional suitability of reinforced concrete for this particular class of building. Some of the points referred to in the article, the circular shape of the auditorium, the freedom from visual obstructions by a bold use of cantilever construction, the concentration upon and converging towards the stage as the focal point, and a quiet and restful scheme of decoration, are all embodied in a theatre, now practically completed, at London Road, Portsmouth, which is being executed entirely in reinforced concrete. It is planned in a novel manner, and contains a number of interesting features in the design and lay-out, so that a brief account of the building may be of interest.

The cinema is being built on a site facing the London Road (*Figs.* 1 and 2), and with access at the back to Croft Road, on to which two of the exits open. The London Road front is the façade of the building and contains shops on either side of the main entrance, leading into a spacious lounge and queue room, from which a wide central staircase gives access to the tea lounge or ante-room, while a staircase on either side leads up to the balcony.

The back of the auditorium proper is on a level with the tea lounge and in direct communication with it, being a few steps higher than the street, while the stage end is, of course, a good deal lower, owing to the ramp. Under the gallery and half-way between it and the floor level, is a mezzanine floor (*Fig.* 3), where the manager's office, staff rooms, chair store (or assembly rooms), are situated. While the arrangement of the component accommodation follows the more or less conventional lines in cinema planning, it is, however, in the auditorium proper that the architect, Mr. A. E. Tutte, of Portsmouth, has introduced a number of original features. Calculated to seat 2,500 persons, the shape of the main hall is circular, with a diameter of 105 ft., and roofed with a saucer dome; concentric with the outer circle formed by the walls is the seating space, divided from the walls by a level gangway leading from the entrance end to the stage end, which is reached by a stair at the end. The whole of this seating space is unbroken by supports, as the main columns carrying the roof are arranged at the corners of a square inscribed in the circle, thereby reducing the span of the roof beams without interfering with the line of vision from any seat.

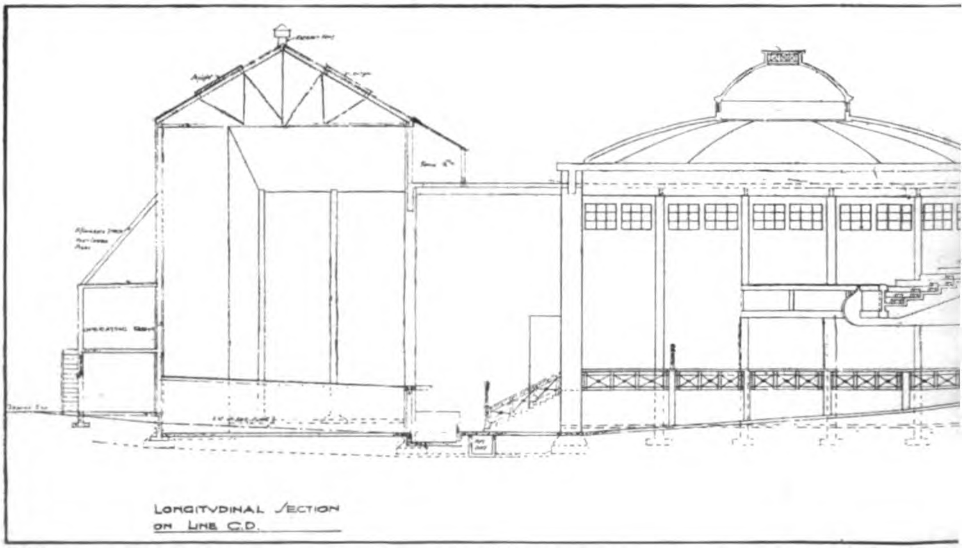
The span of the roof beams is 68 ft. 6 in., and they are formed as T-beams which carry the dome, the roof outside the dome being flat. In the early stages it was not intended to have a balcony, but provision was to be made for it to be added at a later date by spanning a T-beam across the building between two main columns, which latter were to be calculated strong enough to carry the balcony load. It was, however, decided during the progress of the work to include the balcony, which is carried by three main cross beams supporting a number of smaller longitudinal beams. One of the main cross beams is the provisional T-beam already mentioned, which runs under the middle of the balcony, spanning 68 ft. 6 in. like the roof beams and supported by the same columns as the latter. The back cross beam is formed by the wall of the mezzanine floor, spanning 75 ft. and carried by two columns, whereas the front cross beam is formed by the curved parapet of the balcony spanning about 60 ft. While the two former are supported directly by columns, the parapet beam is carried on the tips of two heavy cantilevers from the main roof columns, with anchor beams extending back to the mezzanine beam columns. The parapet beam, on account of its curvature, is stressed both to bending and torsion, and provision is made in the cantilever to take the torsional reaction as well as the vertical reaction, the end of the cantilever being curved outwards and upwards to afford the necessary fixity for this purpose (*Fig.* 4, pp. 48–49). The cantilevers and their anchor beams converge towards the stage so as to follow the general trend of the architectural treatment, which in the whole lay-out has made the point of treating the proscenium as the culminating point of the main hall.



By the arrangement of two main cantilevers carrying the parapet beam, the **T**-beam under the middle of the balcony is relieved of a considerable amount of load, which would have made it deeper than desirable, whereas it is now taken by the parapet beam and cantilevers where sufficient depth is available. As it was not deemed feasible to allow the main column to absorb the cantilever moment, the back extension of the cantilever was introduced, by which means the cantilever moment is taken by the uplift in the mezzanine columns. Under certain conditions of loading, it was found that the dead weight of the mezzanine beam and columns would not be sufficient to counteract the cantilever moment ; or, in other words, the latter columns become ties. To provide for this contingency the foundations for the mezzanine columns were made deep enough and large enough to interest a sufficient body of the filling so that the dead weight of the latter would produce the necessary extra resistance against the uplift. *Fig.* 5, which shows the mezzanine beam, also gives the details of the foundations, from which the arrangement will be clear ; the mezzanine beam, in addition to the balcony, also carries the mezzanine floor.

The balcony itself is formed as a stepped slab in which the risers act as beams, the latter being carried by a number of converging longitudinal beams over the three main cross beams.

The roof slab taking the dome is designed without beams, but the slab outside the four main roof beams is



Fi
REINFORCED CONCRETE

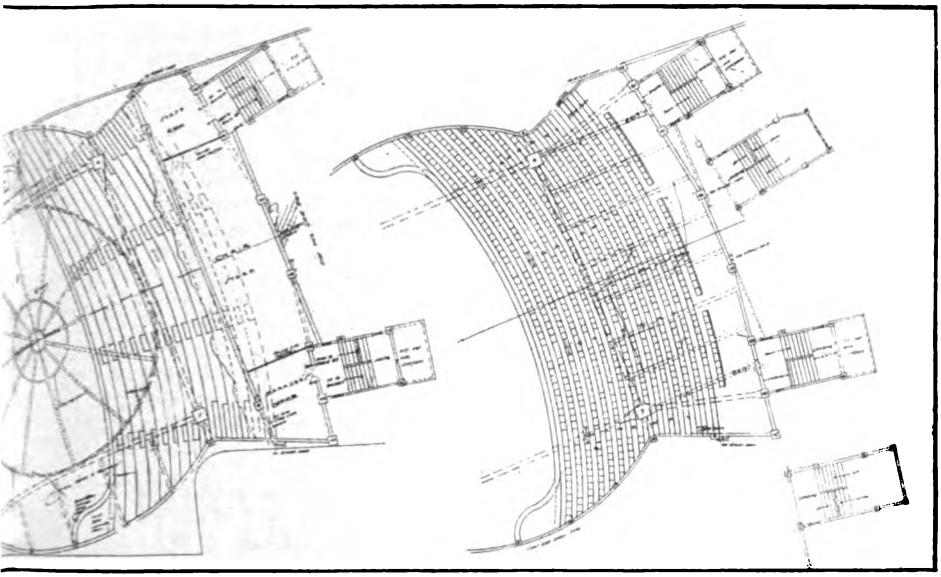
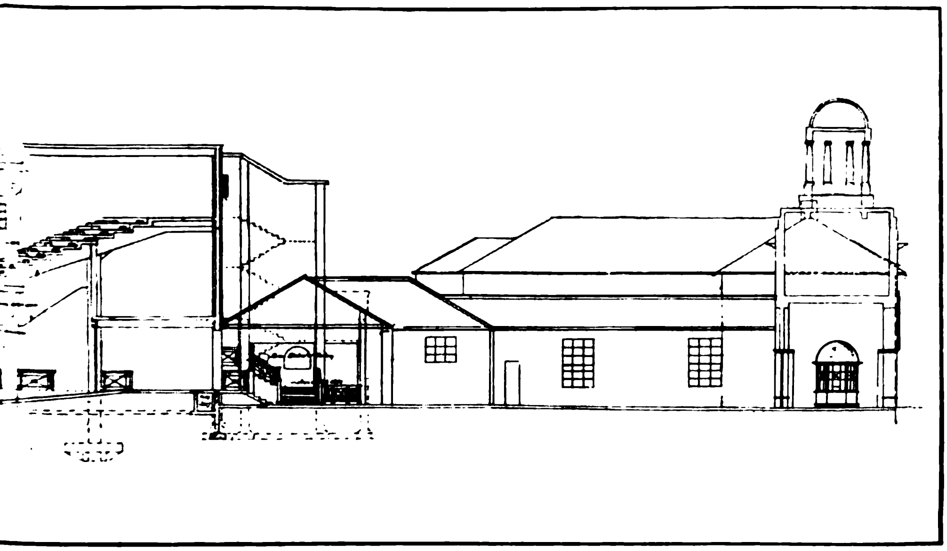


FIG. 2.

formed in the usual way with 4 in. slabs and beams.

Generally speaking, the fullest use has been made in the calculations of the continuity of the structure, and it may be truthfully said that if a load comes on to the balcony, the proscenium columns feel it, and the roof beams respond to it.

Behind the proscenium is a 40 ft. deep and 60 ft. wide stage, it being the intention to use the building for theatrical performances as well; at the back of the stage the operators' room is arranged, a novel point in the arrangement of the



MA AT PORTSMOUTH.

cinema being the back projection, overcoming the difficulty of too long a throw. Under the stage two rear entrances or exits are provided, leading into Croft Street.

The heating and ventilating arrangements are of the most modern type, the air being filtered and washed before passing through the heating apparatus, and driven through a system of ducts under the floor for distribution in the hall.

All the walls are hollow, both the cylindrical part round the main hall and the straight portions, and are carried out in reinforced concrete as panels between columns and stringers, a particularly fine surface having been obtained by the contractors' making use of a special metallic shuttering moving upwards as the work proceeded. Generally, the surface of all the concrete shows such a fine face that no plastering whatever is to be used, the idea being to treat the decoration as a simple colour scheme, without the usual veneer of marble and gold magnificence.

The circular site is specially suitable for chuting the concrete, and the contractors, Messrs. Bevis, Ltd., of Portsmouth, have installed an "Insley" tower and chute in the centre, this plant handling all the concrete for the main hall. The mixture of the concrete for the main columns and their beams was 1 : 1½ : 3, and for the remainder of the work 1 : 2 : 4, the aggregate being dredged by the contractors, and the cement being of the "Ferrocrete" brand, a combination which yielded excellent crushing results.

The whole of the concrete work is now completed, but owing to the plant on the site it has been difficult to obtain satisfactory photographs of the work in progress.

On the instructions of the architect, Mr. R. N. Strover, B.Sc., M.I.M.E., of 4, Westminster Palace Gardens, S.W.1, consulting engineer, designed the whole of the reinforced concrete construction.

REINFORCED CONCRETE BUILDINGS IN BOMBAY.

In May last the Government of Bombay appointed a Committee of engineers to investigate the utility of reinforced concrete for structures in Bombay.

The conclusions arrived at after an inspection of the oldest buildings are:

(*a*) The deterioration of the concrete in every case is traceable either to defective design, inferior materials, or indifferent workmanship.

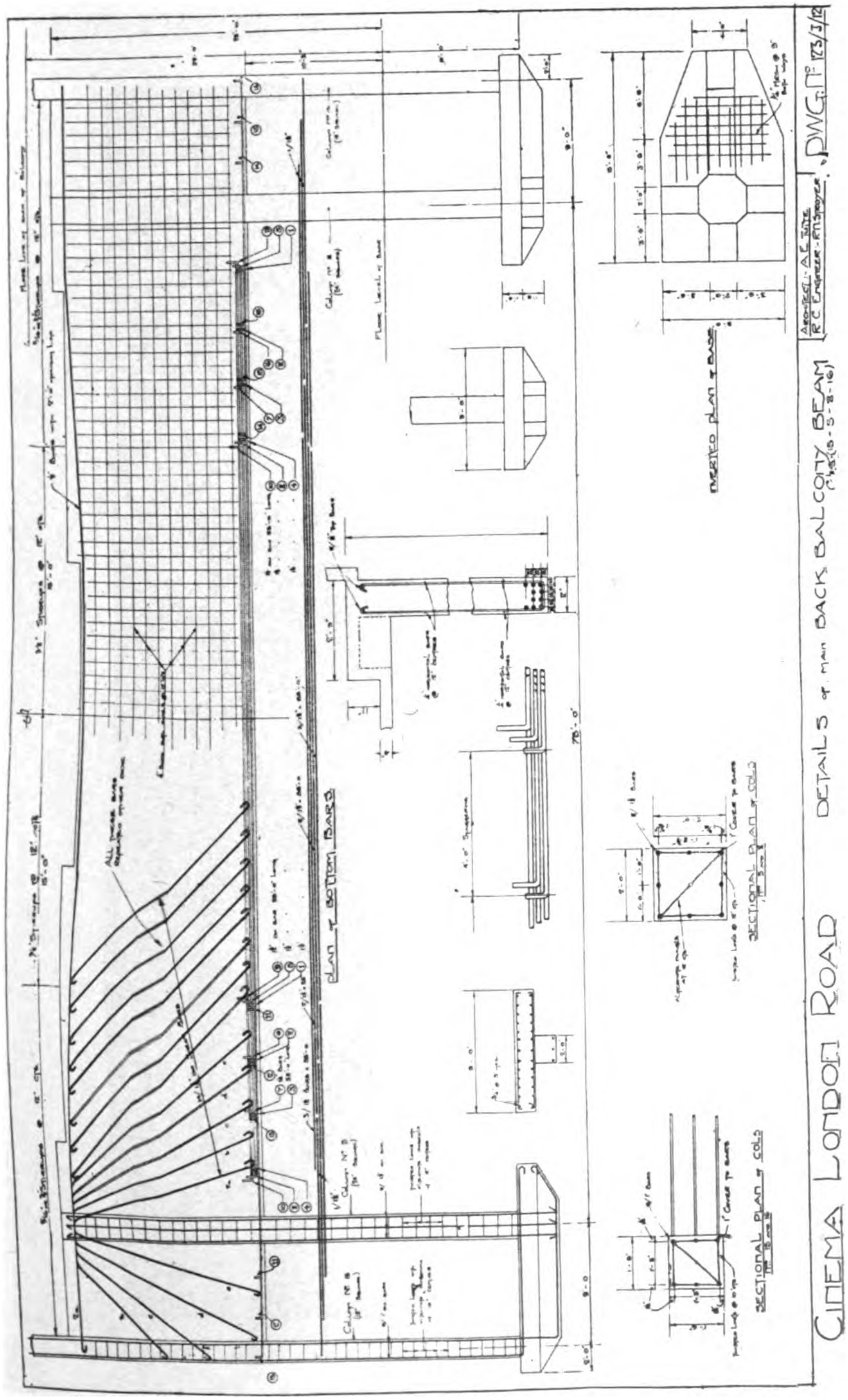
(*b*) The quality of concrete has a direct influence on the preservation of steel since there is no evidence that steel embedded in sound concrete with adequate covering has been adversely affected by the climatic conditions of Bombay. It has also been observed that wherever bad concrete or insufficient covering was in evidence the steel has as a rule corroded. On the other hand, amongst the buildings visited there are a large number of well-designed and carefully-erected structures in which there is no evidence of deterioration of concrete or of steel and which have successfully withstood the rigorous nature of the Bombay climate.

The committee considered other factors likely to be detrimental to reinforced concrete structures, namely (*a*) electrolytic action, and (*b*) the presence of saline deposits in the aggregate.

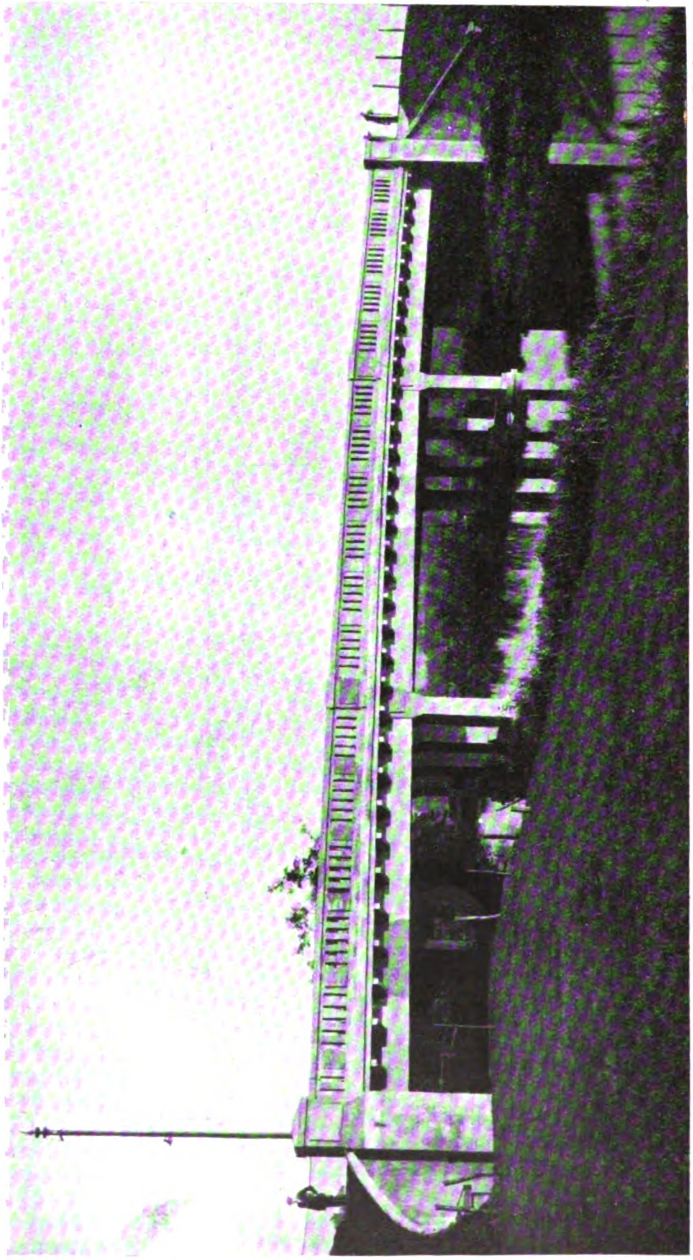
Regarding (*a*), the results of the Committee's inquiry in this connection is that electrolytic action has played no part in producing the defects found in the buildings inspected, and they conclude that in general electrolysis is not to be feared, although it may exist in exceptional circumstances.

In regard to (*b*), the Committee is of opinion that aggregates containing an appreciable percentage of salts are detrimental, but that the shingles and sand in general use in Bombay do not contain sufficient salt to render their use objectionable.

Taking all the evidence into consideration the Committee is of opinion that reinforced concrete structures, if properly designed and carefully executed with sound materials, are pre-eminently suitable for Bombay, and may confidently be expected to be as permanent as the best type of building and more permanent than the average type of construction in use in the city. The Committee is further of opinion that the reinforced concrete structures of Bombay generally do not receive adequate attention, and considers that many of the defects noticed could have been rectified had they been attended to in time.



REINFORCED CONCRETE CINEMA AT PORTSMOUTH. (*See p. 15.*)



[Mr. Ivor F. Shellard, A.M.Inst.C.E., Borough Engineer.

NEW REINFORCED CONCRETE BRIDGE AT TAUNTON. (See p. 21.)

NEW REINFORCED CONCRETE BRIDGE AT TAUNTON.

THE Taunton Council has had under consideration the construction of a bridge at Priory for a number of years, and the matter came to a head in December, 1920, when, owing to the acute state of unemployment in the town, it was decided to proceed with numerous important works, amongst which Priory Bridge was included. The Streets and Highways Committee was authorised by the Council to invite tenders for the construction, and it reported to the Council in February, 1921, that it had carefully considered various designs submitted and recommended the acceptance of Messrs. L. G. Mouchel & Partner's scheme, together with the tender of Messrs. Cowlin & Son (of Bristol) for the construction. It will be of interest to know that over 30 designs and tenders were submitted by engineers throughout the country; invitations were given through the professional journals, and the general conditions of the scheme were set out by the Borough Engineer, Mr. Ivor F. Shellard, A.M.Inst.C.E. Designs submitted included almost every form of bridge structure, and the Committee, after very careful consideration, adopted Messrs. Mouchel's design on account of its workmanlike appearance and also because it gave a very large water area, which was a very important consideration.

By the photograph which we give it will be seen that the bridge is of three spans, the two end spans being 42 ft. 6 in. and the centre span 50 ft., giving a total length between the abutment walls of 135 ft. It is built throughout in reinforced concrete, and the foundations both for the abutment walls and the centre piers rest upon the marl found at a depth of approximately 14 ft. below the towpath level.

There are nearly 80 tons of steel in the bridge made up of hundreds of bars ranging from over 2 in. in thickness to steel wire of less than $\frac{1}{16}$-in. gauge. The work of bending the bars to the required shapes and making hundreds of steel stirrups was done entirely on the site, the local unemployed labour after tuition showing much adaptability for the work.

The width of the bridge between the parapet walls is 40 ft., made up of two 8-ft. pathways leaving a 24-ft. carriageway. There are four big main beams each measuring 2 ft. 11½ in. by 18 in.; these rest on four piers situated in each abutment wall and four piers situated in two positions in the river. The piers in the abutment walls are carried right up from the marl level in solid concrete; four piles (14 in. and 12 in. square) were driven down into position underneath each pier in the river, and around these piles a concrete cylinder was placed and sunk into the river-bed to a depth of approximately 1 ft. Each cylinder, which formed a circular cap to the four piles, was then filled in with solid concrete; steel rods were placed in position, and the piers were built up by making wooden moulds and then filling in with concrete. The main beams support the secondary cross beam, each 16 in. by 7 in., placed at

NEW REINFORCED CONCRETE BRIDGE AT TAUNTON.

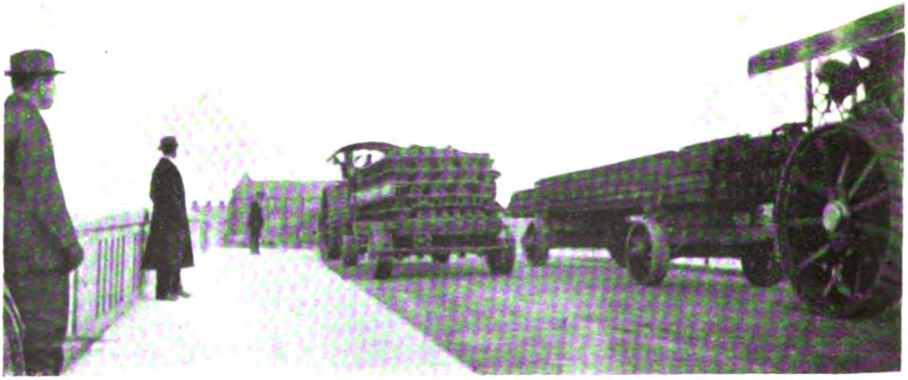
approximately 4 ft. centres, and the whole is covered by a raft of concrete 6½ in. in thickness. Every beam, and the concrete slabs, piers, and abutment walls (each of which, by the way, is only 6 in. thick) are built up with hundreds of steel bars of the various thicknesses consistent with the stress each member has to take, and from the commencement these bars were interwoven, bent into the various positions required, and locked together as the work of concreting gradually proceeded, so that from the foundation to the top of the parapet wall there is continuity throughout.

The contractors commenced operations in July, 1920, and completed the work in July of this year, well within the contract time.

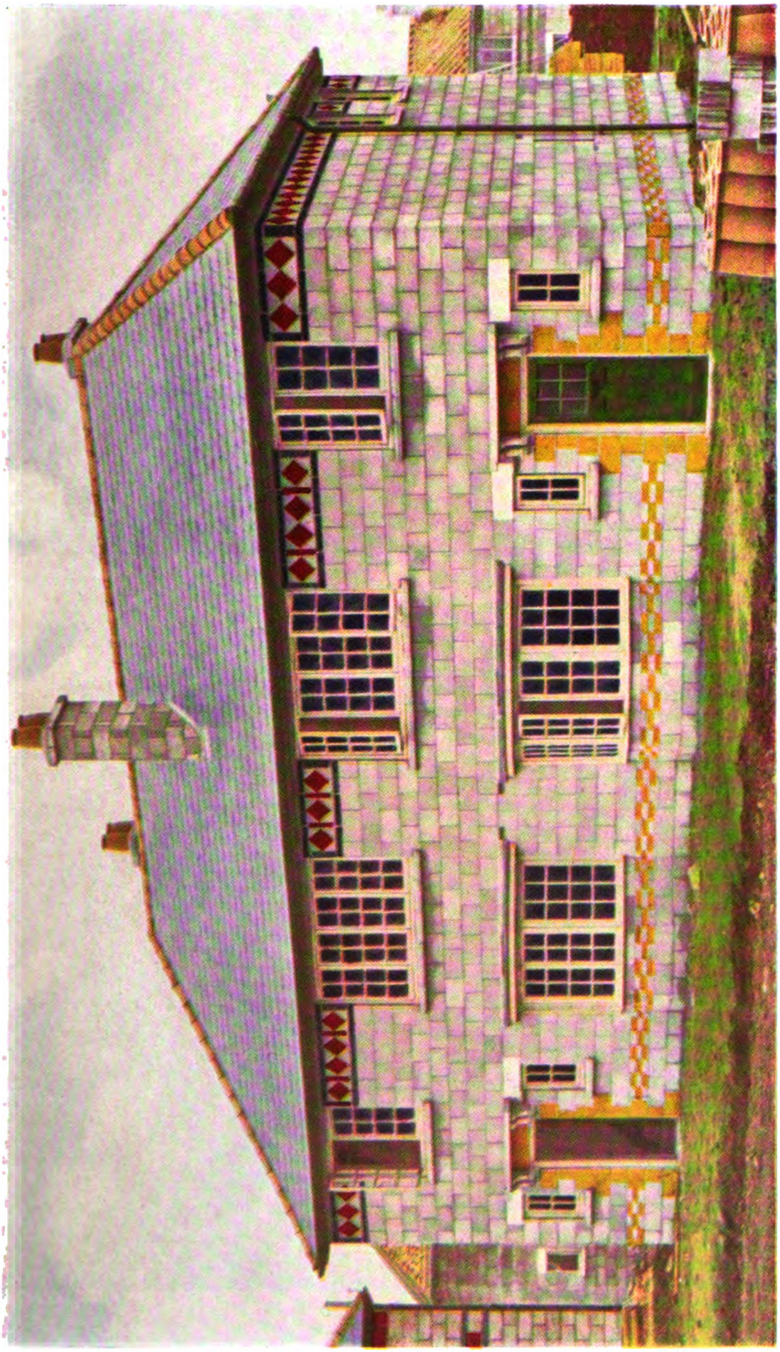
The bridge was allowed to stand for three months before traffic was allowed on it, thus allowing the concrete to mature and harden before the passing of loads at the end of the three-month period. The bridge was tested under the conditions laid down by the Borough Engineer, and for which the bridge has been designed. The general conditions provided for the test to be as follows:—"When the bridge is completed and before traffic is allowed on same a test load shall be passed over the bridge in the following manner:—A traction engine weighing 16 tons on a wheel base of 11 ft. by 6 ft. or thereabouts shall pass over, followed by a boiler trolley with a total load of 40 tons. In like manner a similar load shall proceed from the opposite direction and at the same time the two shall pass on the bridge. This shall be repeated at various positions on the bridge at the option of the Engineer. Further, each load shall be immediately followed by a 15-ton steam roller, and during the test the footpath on each side shall be loaded over the whole of one span or some portion of each span (at the option of the Engineer) with a load of 200 lb. to the super foot, the whole of the above loads to be in addition to the weight of the structure. Should the Engineer direct, the whole of the above loads shall be thrown alternatively on one side or the other or the centre of the bridge. During the period of testing the deflection shall be carefully noted and accurate records kept."

The photograph on this page shows the most severe test being carried out. The loads passing each other on the bridge each has a weight of 40 tons on two axles and drawn by a 16-ton traction engine in each case; thus the total load on the bridge under this test was 112 tons. Other methods of loading were applied, such as loading the footpath and passing different loads over the bridge, and in order to subject the bridge to as severe an impact load as possible whilst 40 tons were loaded on one end a traction engine pulling 20 tons was sent over at a speed of about 10 miles per hour. The tests proved satisfactory in every respect, the maximum deflection in no case being more than $\frac{1}{9000}$ part of the span and each beam entirely recovering after the passing of the loads.

A feature in the complete bridge is a niche or hole in the north-west abutment



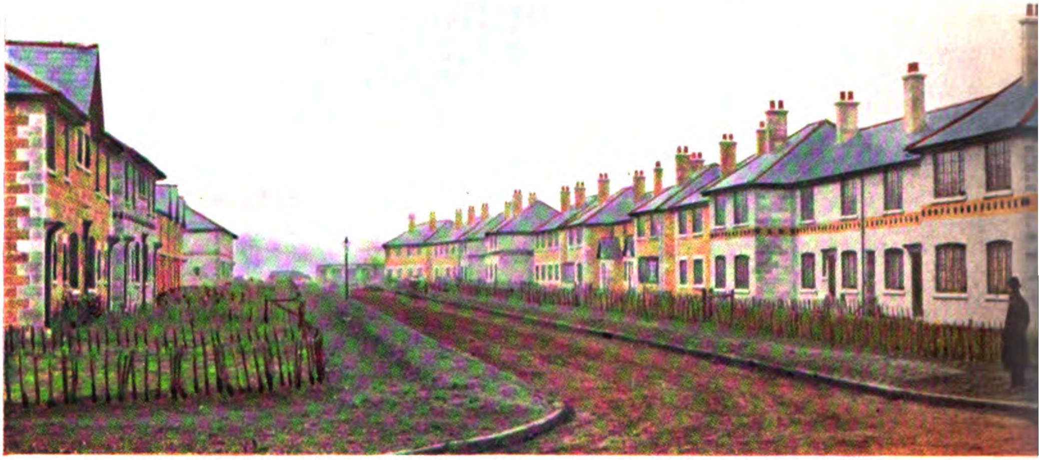
New Reinforced Concrete Bridge at Taunton: Under Test.

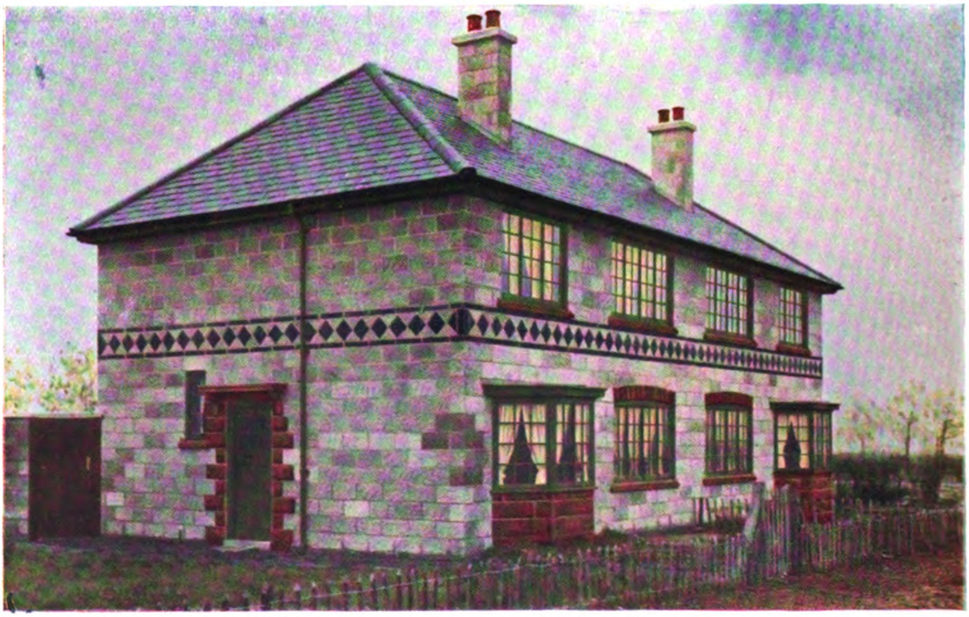


COLOUR IN CONCRETE.

Architect: F. E. G. Badger, A.M.I.C.E.

Pair of Houses in Concrete Blocks on the Cherry Lane Site of the Liverpool Housing Scheme.





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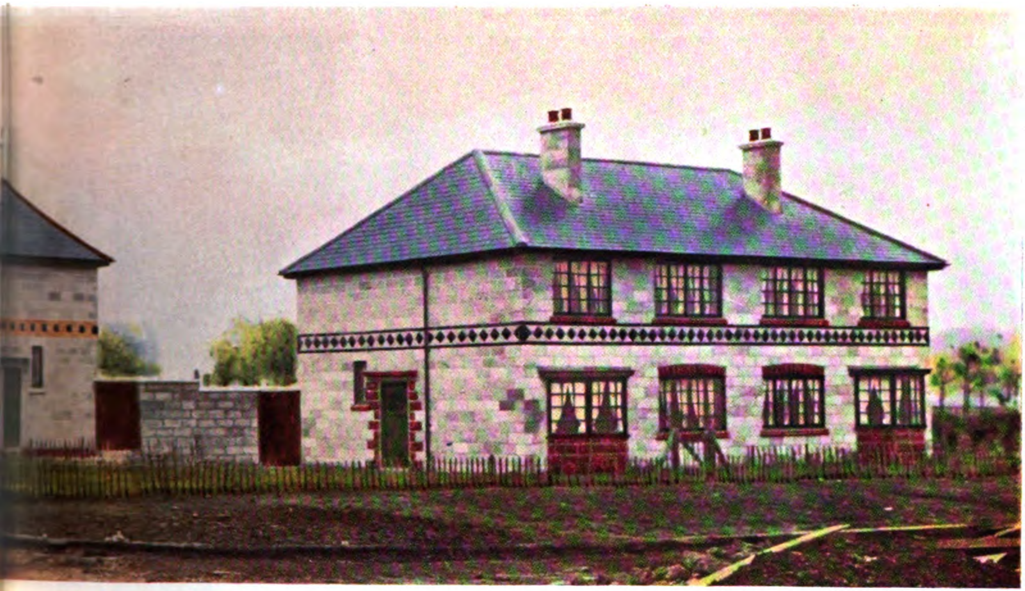
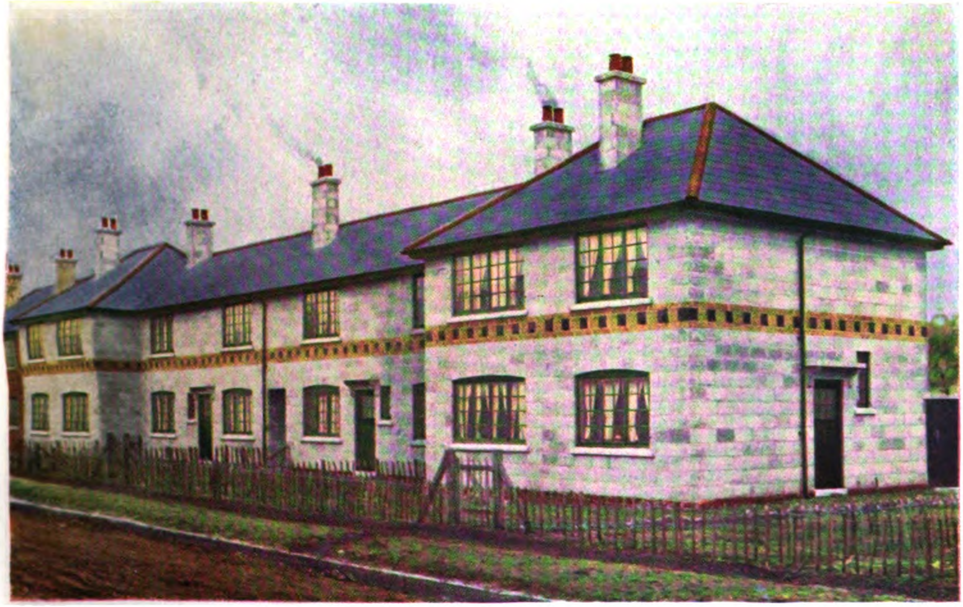
Some Views on the Cherry Lane Site of the Liverpool Housing
(Architect: F. E. G. Badger.

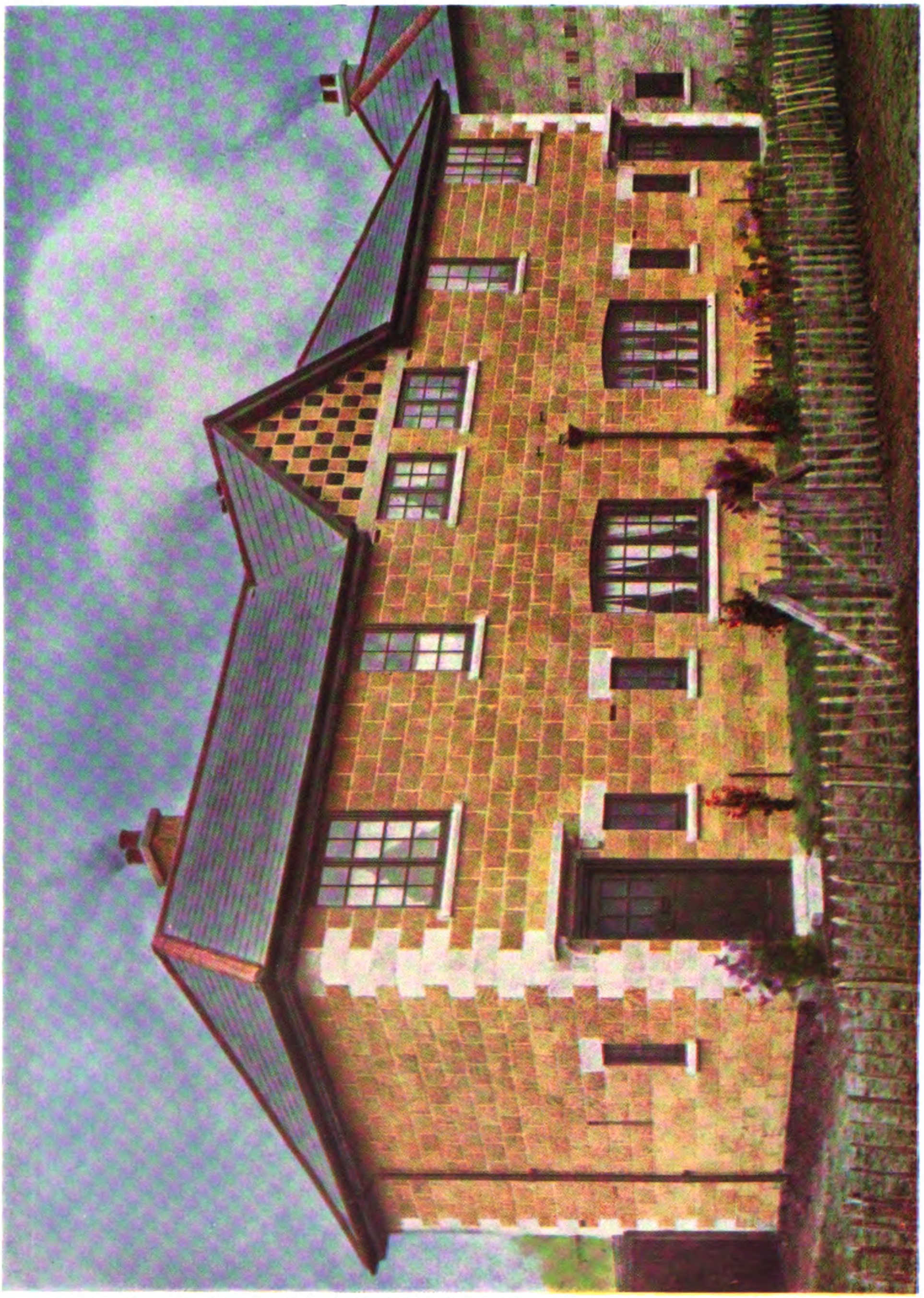




ONCRETE.

ere several thousand Houses are being built with Concrete Blocks.

E. Director of Housing, Liverpool.)



COLOUR IN CONCRETE.

Pair of Houses in Concrete Blocks on the Cherry Lane Site of the Liverpool Housing Scheme.

Architect: F. F. G. Badger, A.M.I.C.E.

in which is placed an airtight tin containing the complete plans, working drawings, general conditions, and specifications, with a description on parchment. The tin rests in the niche, which has been completely sealed up in the concrete, a small bronze plate on the face denoting the fact. Mr. Shellard's idea in doing this was to facilitate matters for the engineer who, perhaps in a hundred years' time, would have to extend the bridge. On the opposite abutment is a bronze tablet giving the date of the bridge and the names of those responsible for its erection. This was presented and placed into position by the contractor, and is the work of Messrs. J. W. Singer & Sons, of Frome.

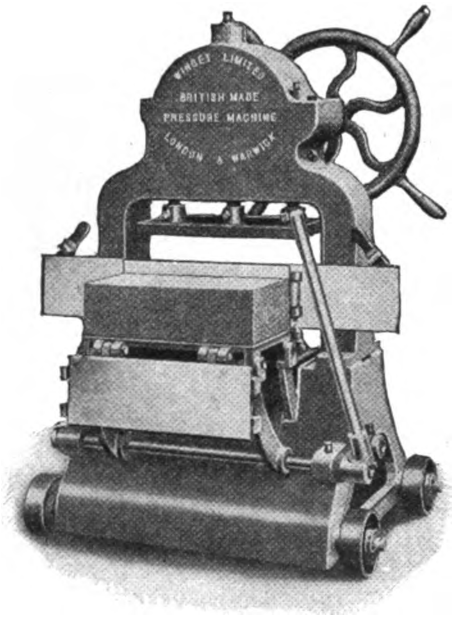
The whole of the work from start to finish has been carried through under the direction of the Borough Engineer, Mr. Ivor F. Shellard, A.M.Inst.C.E.

COLOURED CONCRETE ON THE LIVERPOOL HOUSING SCHEME.

THE coloured concrete houses illustrated in the supplement given with this issue have been erected on the Cherry Lane Estate of the Liverpool Corporation, under the direction of Mr. F. E. G. Badger, A.M.Inst.C.E., Director of Housing. Already 238 working-class houses with coloured exteriors have been built or are in course of erection on this site.

The houses are built of machine-made concrete blocks, each block bearing the appearance of having received special individual attention by reason of the external markings differing in every case. The aggregate used for blocks which are not to be exposed externally is furnace clinker, screened, crushed, and graded, the proportion by bulk being five of clinker to one of Portland cement. Blocks which are to be exposed externally differ from the internal blocks only in having a facing about 1 in. thick—before pressing—of hard-aggregate concrete composed of three of aggregate to one of cement, the two mixtures being cast together in one operation. The aggregate in the facing may be sand only, or gravel, or crushed granite, or other suitable hard stone, and with it is incorporated specially-made colouring matter; so that any colour of block may be produced either by showing the natural aggregate or by means of the added pigments. Variety in surface texture is obtained by stippling the block with the same mixture as the 1 in. facing material, except that the aggregate itself should be a little smaller in size. A small quantity of iron oxide is thoroughly incorporated so as to produce any desired colour. The stippling is done while the blocks are "green," and it is important that the stippling should not be delayed much beyond what is found from experience to be the proper time, as it is largely by this means that an intimate incorporation with the main substance of the block is ensured. The stippling is not, therefore, merely an overlaid coating. The walling and partitions throughout the whole of the houses are constructed of concrete slabs, 2, 3 or 4 in. in thickness, according to the position they occupy and the load to be carried. The external walls are 9 in. thick, formed of two leaves, the outer leaf, or skin, of 4-in. faced slabs, and the inner leaf of 3-in. slabs, with a 2-in. cavity between; the two leaves are linked together at intervals with tarred or galvanised iron ties. Party walls are 8 in. thick, with two 3-in. leaves and 2-in. cavity, and are also tied together. Partition walls which are two stories high, or which though only one story high carry floor joists, are of 4-in. slabs, and single-story partitions which do not carry floors are of 2-in. slabs. The slabs are set in mortar by bricklayers or masons working from a scaffold in the usual way. The continuous cavity in the external walls renders the houses dry, cool in summer and warm in winter, as the air in the cavity acts as an insulator.

The main walls so far have been treated with various shades of buff colouring; also grey, white and red, with string courses and cornices picked out in black and other shades according to the colour scheme of the main walls. Quoins of a different tone can be introduced according



Block-making Machine as used on the Liverpool Housing Scheme.

to taste, without any extra cost, in the external angles, under bay windows and at the sides of entrance doors.

With the use of this type of block (called the "Nonalike") no two houses are exactly alike, yet they blend together remarkably well. Houses which have already weathered two winters on the Cherry Lane site testify both to their absolute dryness and to the durability of the colouring.

Mr. Badger gave much consideration to the problem presented by the difficulty of obtaining plasterers, and decided to dispense with them altogether by building the interior walls as follows :—Sand-faced concrete blocks, 18 in. by 9 in., are used to form the interior walls. They are set with a flush mortar joint, and a few days afterwards are gently rubbed down with a soft stone. Ordinary wallpaper is applied in due course, and the only precaution necessary is to add a little size to the paste, a coat of which is given to the wall itself in addition to the usual dressing on the back of the paper. If a wall-paper is selected with a fair amount of pattern, such as the "tapestry" variety, it is claimed that a satisfactory result is certain. A still further use to which the coloured blocks have been put is in the forming of a dado of, say, a red colour, with grey or cream-coloured blocks above, which may be left without any further treatment ; or the dado may be sized and varnished.

All the houses on the Cherry Lane site were built with concrete blocks made on "Winget" Pressure Machines, some fifty of which, as well as chain-spade mixers and other "Winget" plant, are being used on the Liverpool Corporation housing scheme. The same type of machine is being used on the L.C.C. housing scheme at Becontree, as well as at Glasgow, Brighton and many other undertakings up and down the country. This machine was specially designed for the rapid and economical erection of cottages and small housing schemes. It makes various kinds of blocks, half blocks, and partition slabs, all produced on one kind of pallet, and is very easy to operate. It has been considerably strengthened and improved in the latest type, incorporating the results of the experience in scientific blockmaking in all parts of the world.

Concrete in the Repair of Ancient Monuments.

In an appeal for £5,000 to put the spire and tower of St. Mary's, Shrewsbury, into thorough repair, an architect's report states that unless stringent remedies are carried out without delay a catastrophe may take place, and the occurrence of nearly thirty years ago, when part of the spire fell through the roof of the nave is recalled. The stonework of the tower has lost most of its original strength since Norman times, and it is proposed to insert a continuous belt of ferro-concrete at the foot of the spire, grouting up with cement all the loose and disturbed portions of the masonry, and replacing all the worst weather-worn stones with new. This must be followed by a second ferro-concrete belt, and by forcing in cement grout under hydraulic pressure.

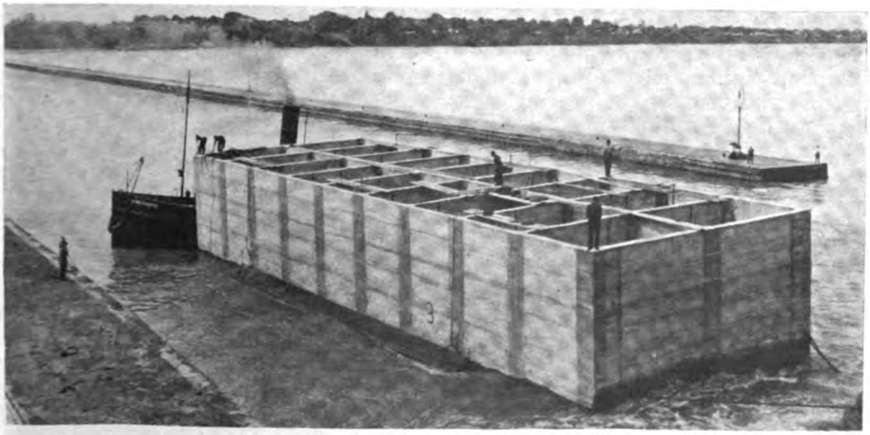
THE WELLAND SHIP CANAL.

THE Welland Ship Canal, now on the way to completion after interruptions caused by the war and labour troubles, follows the valley of the Ten Mile Creek, Ontario, between its mouth, about 3 miles east of Port Dalhousie, and Thorold, and consists partly of the reconstruction of the existing canal and partly of new work. Between Thorold and Allanburg a new cut will be made for the purpose of straightening the alignment of the canal between these points. The Ship Canal will again cross the present one below Lock No. 25, where the water levels of both are at elevation 568.0, the extreme low-water level of Lake Erie above sea level. From Allanburg to Port Colborne, on Lake Erie, it will follow closely the course of the present canal.

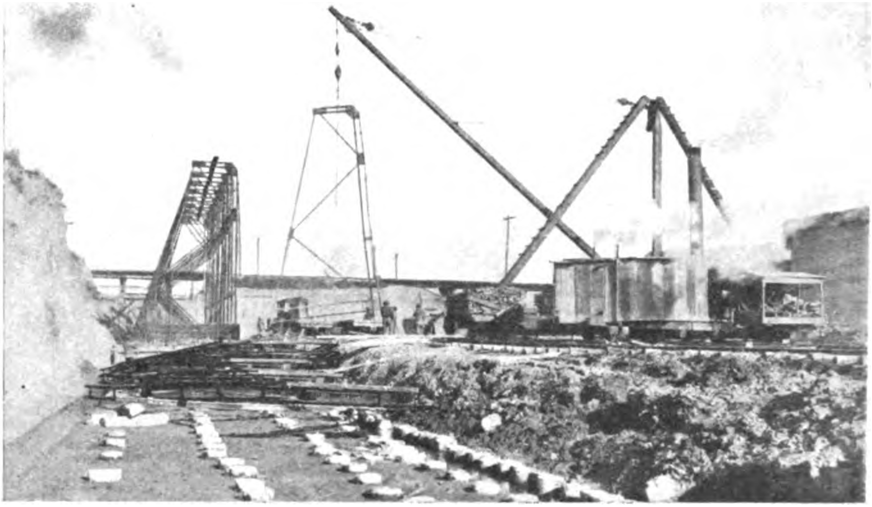
The total length of the canal will be 25 miles, and for all practical purposes of navigation it is a straight line throughout, 2½ miles shorter than the present canal. The fall of 325½ ft. between the lakes is overcome by seven locks of 46½ ft. lift each. The direct line of the canal down the face of the escarpment and the topography of the lower plateau permitted the adoption of these high lifts, which constitute a feature in the design of the canal which has no precedent in actual construction for locks of their size. The few locks required permit a rapid descent into Lake Ontario, and diminish the time required for vessels to pass through the canal. The usable dimensions of the locks are 800 ft. long, 80 ft. wide, with 30 ft. depth on the sills. The canal will be 200 ft. wide on the bottom, with 2 to 1 slopes, and for the present will be excavated to a depth of 25 ft. only. All structures, however, will be built for 30 ft. depth, so that the canal at a future date can be enlarged by simply dredging out the canal prism and harbour entrances.

The above dimensions of the canal prism give a wet section of about four times the immersed section of a ship of 75 ft. beam and 22 ft. draught, and when completed to the 30-ft. depth the ratio of channel to the immersed section of the largest ship will be about 3½.

At the Ontario end of the canal Port Weller Harbour is being formed by two earth embankments extending 1½ miles out into the lake, where the depth is 30 ft. at extreme low water. The banks, which are being formed from the materials excavated from the canal prism, protected on their inner and outer slopes with rock, are parallel to one another, and provide a channel 800 ft. wide and 5,000 ft. long, which, at the entrance, is narrowed to 400 ft. wide by two converging lines of reinforced concrete crib work. From the shore the harbour channel extends inland about ½ mile, gradually narrowing from 800 to 200 ft. in width to Lock No. 1, 9,500 ft. from the harbour entrance. The harbour docking is being built of reinforced concrete cribs. The docking



WELLAND SHIP CANAL: REINFORCED CONCRETE CRIB.



WELLAND SHIP CANAL: PLACING STEEL FRAMES FOR UPPER ENTRANCE WALL, LOCK NO. 1.

on the west side of the harbour is connected with Lock No. 1 by a long entrance pier of reinforced concrete. Above Lock No. 1 the canal follows the bed of the creek to Lock No. 4. Lock No. 2 is situated about 1½ miles above Lock No. 1, where the rock surface was fortunately found at an elevation slightly below that required for the lock foundations. Lock No. 3 is located just north of the present canal, or about 2½ miles south of Lock No. 2. The rock at this point is about 20 ft. lower than required for the foundation of the structure.

Between the guard gates and Allanburg a new channel, on a direct line, will be excavated, and the present canal between these points abandoned. From Allanburg to Port Robinson the present canal is being widened on the west side and deepened for 25 ft. draught.

Between Port Robinson and Welland it is proposed, instead of straightening and enlarging the present canal and building a new aqueduct to carry the Ship Canal over the Chippawa at Welland, to raise the level of the river permanently about 6 ft. to the summit level of the canal (elevation 568.0), by means of a dam across the river at Port Robinson, which would be provided with stop-log sluices for allowing surplus waters to flow through the dam back into the river and out into the Niagara at Chippawa, as at present. The river valley would then be used as the canal channel from Port Robinson to Welland and the present aqueducts dredged out.

From Welland to Ramey's Bend the present canal will be deepened and widened along its western bank, and a new cut made across the point that forms the bend for the purpose of straightening the canal opposite the point. From Humberstone, immediately south of Ramey's Bend, to Port Colborne the Canal will be deepened and widened on the west side.

The improvements to Port Colborne Harbour will consist of deepening the inner and outer harbours to the 25 ft. depth, and the entire removal of the old guard locks and supply weir at the north end of the Harbour. A new guard lock will be built in the new cut north of Humberstone, and a supply weir built across the section of the present canal, which will be abandoned, at Ramey's Bend. It is also proposed to build a new western breakwater, extending 2,000 ft. farther out in the lake than the present one, for better protection to the Harbour during high gales. The new breakwater would consist of a rock bank built from the rock excavation north of Port Colborne.

There will be thirteen highway bridges, three combined highway and electric railway bridges, one electric railway bridge, and five steam railway bridges, across the canal. These bridges will have 80 ft. clear spans where they cross the foot or head of locks, and 200 ft. clear spans where they cross the canal prism. They will all be bascule bridges, except where local conditions will permit the economical and satisfactory use of swing bridges.

REINFORCED CONCRETE CRIBS.

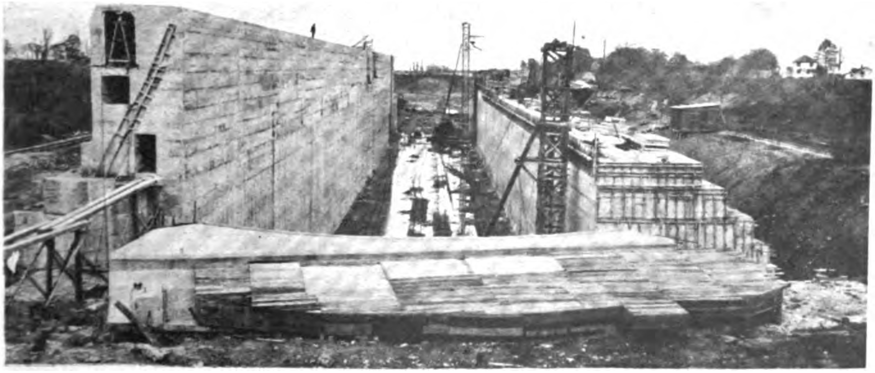
The standard cribs are 110 ft. long, 38 ft. wide, and 34 ft. high, divided into 18 compartments by 1 longitudinal and 8 cross walls. The front and rear walls vary from 12 in. thick at the top to 18 in. at the bottom, the end walls from 15 in. to 21 in., and the longitudinal and cross walls from 10 in. to 14 in. Along the face of the cribs a heavy wall about 7 ft. high is built of reinforced concrete blocks and mass concrete. The blocks are 9 ft. long, 9 ft. 3 in. wide, and 3 ft. high, and are supported at the face by the wall of the crib and at the rear end by concrete beams built between the cross walls of the crib. Above these blocks the wall is finished to coping level in mass concrete.

A standard crib contains 934 cu. yds. of concrete and 130,000 lb. of steel rods (¾-in. diameter), and weighs approximately 2,000 tons. Each crib is built to its full height on a floating pontoon about 9 ft. high, and so constructed that when the crib is finished the sides and ends can be released and allowed to float away, while the heavily-ballasted bottom sinks to the bottom below the crib. Before the pontoon is released from the crib temporary wooden bottoms are fitted in each compartment for the purpose of floating the crib into position on removal of the pontoon. When the crib has been sunk, by filling it with water through valves in the temporary bottoms, the latter are released and recovered for use again. The crib is then filled with shale and rock from the harbour excavation.

When the crib seats have been dredged out, three rows of stone broken to macadam size are deposited through a pipe on the lines of the face, back, and centre longitudinal walls for the purpose of providing as nearly as possible a level foundation for the crib to rest upon when sunk in position. This method of building and sinking the cribs has proved satisfactory. Altogether forty-six cribs are required for the Port Weller Harbour docking.

WEIRS.

At Locks 1, 2, and 3 regulating raceways will be built on the east side of the locks for controlling the service flow of the canal and emptying the reaches when required. The raceway will consist of a concrete lined channel the level of which will be about midway between the upper and lower levels. The flow through the raceway will be controlled at the upper end by a dam, with a series of large valves in its base for emergency use and for emptying the upper pool. Normal regulation will be obtained through shallow stop-log sluices on the crest of the dam. The dam controlling the lower end of the sluiceway will be a stop-log dam with deep sluices.



Welland Ship Canal: Lock No. 1.

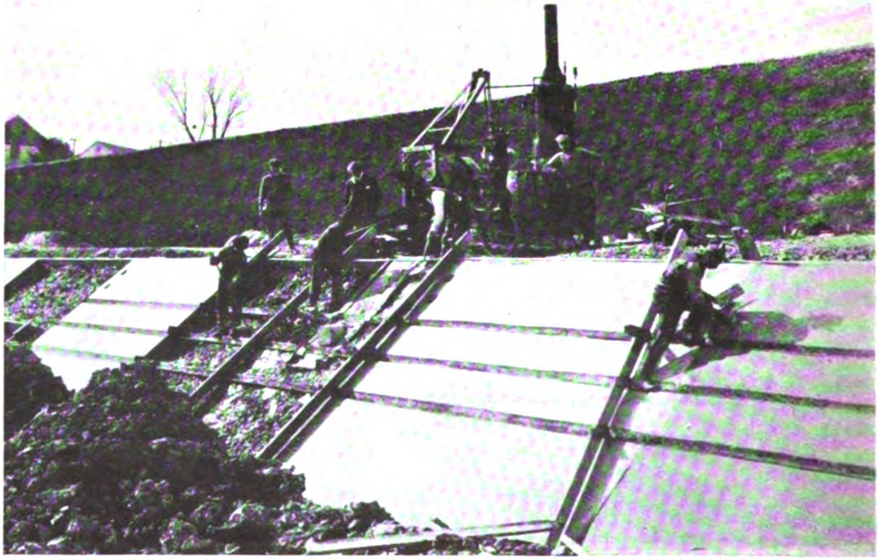
WELLAND SHIP CANAL: HEAD OF LOCK NO. 6.

COST.

The estimated cost of the Ship Canal in 1912 was 50,000,000 dollars, and the expenditure to March 31, 1920, including surveys, engineering expenses, and right-of-way was 20,270,436 dollars. The quantities of several of the main items involved in the construction of the Canal are as follows: —

Earth excavation, 38,500,000 cu. yds.
Rock excavation, 6,600,000 cu. yds.
Watertight embankments, 3,400,000 cu. yds.
Concrete, all kinds, 2,600,000 cu. yds.
Steel reinforcing, 16,500,000 lb.

For the purpose of construction the canal has been divided into nine sections. Owing to the European war the work on



WELLAND SHIP CANAL: CONCRETE PROTECTION OF SIDES.

the canal practically ceased in 1916, and in 1917 the contracts were cancelled and the work entirely closed down. After the Armistice the Government decided to resume work on a small scale, and arranged for the work to be proceeded with by the former contractors: The Dominion Dredging Company (Section No. 1); Messrs Baldry, Yerburgh and Hutchinson (Section No. 2); Messrs. Doheny, Quinlan & Robertson (Section No. 3); and the Canadian Dredging Company (Section No. 5). The work has accordingly been continued in a more or less intermittent manner, due to labour troubles and other causes, since January, 1919.

The engineering work is under the direction of Mr. W. A. Bowden, M.E.I.C. (Chief Engineer, Department of Railways & Canals), and Mr. Alex. J. Grant, M.E.I.C. (Engineer in Charge at St. Catharines).

BOOK REVIEWS.

Materials of Construction. By H. E. Pulver.

London: McGraw-Hill Book Company. 318 pp. + xviii. Price 15s.

This is an excellent treatise for the use of students who require a good elementary training in the various materials used in construction, as it is written in a very clear manner and the essential points are covered by the author in a straightforward way which enables these points to be gathered without the necessity of perusing a mass of unimportant detail.

Plasters and natural and artificial cements are first dealt with, and the uses and application to building work are then given. Concrete is considered in a good chapter which covers the proportioning, mixing, placing, and properties in a thorough manner, and this is followed by notes on building stone, brick and other clay products, and timber. Chapters devoted to various metals are then given, and miscellaneous materials of minor importance are briefly described in the last chapter.

Three appendices complete the volume, but as these deal with American standards they will not be of much general use to the student in this country. The author does not claim to have covered the whole subject in an exhaustive manner, but has rather endeavoured to deal with the question of materials in a volume which is of such a size that its entire contents may be covered by a student in the length of time usually assigned to the subject in ordinary technical schools, and in this endeavour he has succeeded admirably.

The text is divided up into 380 articles, which are numbered and clearly indicated with a sub-heading, and thus reference to any particular point is a simple matter. The volume can be thoroughly recommended to the student in building work as a very useful and reliable textbook which will enable him to become acquainted with an important subject with the minimum amount of study.

A. L.

Sea-going and other Concrete Ships. By N. K. Fougner.

London: Oxford Technical Publications, Henry Frowde and Hodder and Stoughton. 216 pages; 58 illustrations. Price 21s. net.

The use of concrete for building ships reached a climax during the war, since which time most of the yards building these vessels have been closed. This does not mean that concrete is unsuitable for shipbuilding, but merely that the additional yards could not compete with older ones with better commercial facilities. In countries where the facilities possessed by the great British shipbuilders are not available, and especially in remote lands in a primitive state, concrete offers a large and profitable field of usefulness as a shipbuilding material.

The author was one of the first engineers during the war to start building concrete ships, and during the past five years he has designed thirty such vessels. He claims that concrete has a distinct advantage over steel when the cargoes are bulky and measure 70 cu. ft. or more per ton weight, because a concrete boat is heavier and therefore has a larger capacity and a larger space available for deck cargo than a steel ship. He claims also that concrete ships are cheaper to build, cost less for upkeep, are more quickly and cheaply repaired, are not subject to corrosion, have better insulating properties for cargoes, such as ice, fruit, etc., and when empty require less ballast to enable them to ride easily on rough seas. The speed is the same for ships of equal dead-weight and engine-power.

The author deals, in turn, with a number of concrete ships (some of which have previously been described in *Concrete and Constructional Engineering*), makes various comparisons between concrete ships and steel ships, with the results indicated above, and then describes in broad outline the preparation of the concrete, the arrangement of the reinforcement, the conditions governing design, and the general method of construction of ships. A short chapter on special details introduced into various ships is also included, and is followed by a number of recommendations by the author.

It is interesting to observe that the author ensures the waterproofness of the concrete solely by carefully grading and proportioning the mixtures, and not by the use of any waterproofing agent, though he suggests an external coating of hot coal-tar below the water-line " to make still more sure " and to prevent marine growths.

The author candidly admits that the methods of design and construction hitherto used are far from perfect, but considers that modifications are merely the natural course of development. Would-be designers will be greatly assisted by a complete set of calculations, " just as they were made in the office," of a trial ship, the *Askelad*, a 1,000-ton dead-weight cargo vessel, propelled by two 160-h.p. Bolinder crude oil engines, designed and built at Moss, in Norway.

The Design of Masonry Structures and Foundations. By Clement C. Williams.

London: McGraw-Hill Book Company. 555 pp. + viii. Price 25s. net.

The title of this volume is likely to be misleading to many readers in this country as the author is an American and the term " masonry " is used in America to cover not only stone masonry but also brickwork and concrete, and a large portion of the text is devoted to reinforced concrete work which would not be considered as coming within the scope of masonry work by those unfamiliar with the term as applied in the U.S.A. The book is somewhat different from many which deal with this subject, because the author has devoted a good deal of matter to the consideration of satisfactory architectural treatment of engineering structures, and while methods of calculation are presented they are not given with the idea of covering all the complicated and alternative questions and variations that may arise in the course of engineering practice. Chapters are given on the general principles of construction, the use of stone masonry laid in mortar, brickwork, and plain concrete, and some useful notes are given.

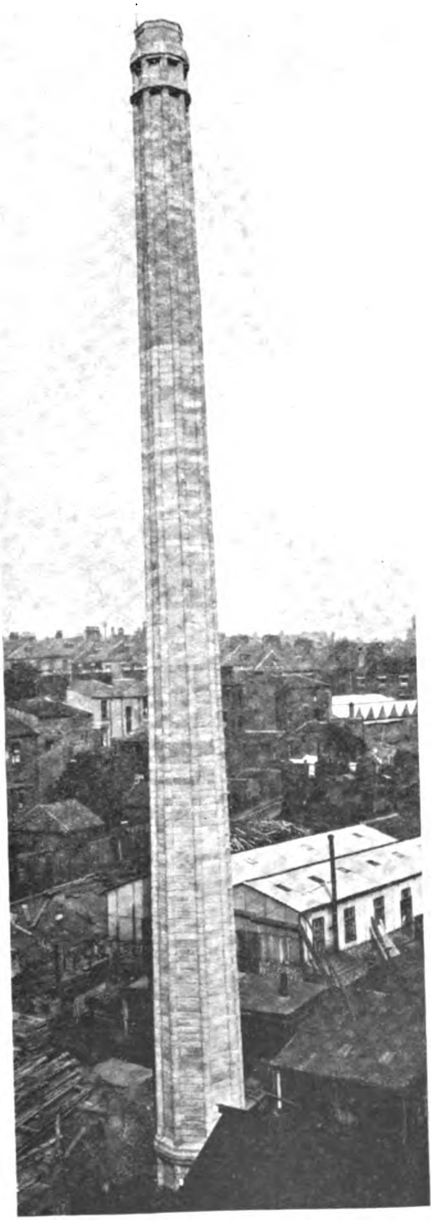
In dealing with the chuting of concrete, however, the author, while describing the difficulties of securing a consistent material, does not mention the use of a receiving hopper at the foot of the chute from which the concrete is taken by " buggies " to the place of deposit, which is the only satisfactory method, but he mentions the transport by buggies as implying an alternative method to chuting only. With a well-organised lay-out and proper equipment the use of spouting for concrete work does not demand exceptional supervision as compared with other methods. The chapter on reinforced concrete work covers the general principles and design, and beams, columns and slabs are dealt with. Other chapters are devoted to arches, sea walls, retaining walls, bridge abutments, viaducts, culverts, bins, and chimneys, and a great deal of matter is covered under these headings. A very useful chapter is given on formwork, but it is rather a pity that metal forms have been ignored in what is obviously a modern text-book. The only mention made of this type is in one sentence, where it is stated that round columns are almost always moulded in steel forms. In view of the importance of metal forms at the present time a few general notes and details in connection with their use would undoubtedly have been advisable. The last three chapters are allocated to the consideration of foundations, and the author states that this section of the work was reserved until the loads transmitted by the superstructure had been studied. It is somewhat unusual to present the matter in this order, but in some respects it is more logical because the foundations cannot be designed and calculated until the superstructure loads have been determined. The volume contains much of interest, and can be recommended as a useful addition to the student's or engineer's library.

A. L.

CONCRETE CHIMNEY IN PRE-CAST SEGMENTS.

We illustrate herewith a reinforced concrete chimney recently constructed in pre-cast segments on what is known as the "Monnoyer" system of chimney construction, which has been extensively used during the past sixteen years in various countries, more particularly in France and Belgium, where this method was originated.

This chimney was constructed for the Co-operative Wholesale Society, Ltd., at Newcastle-on-Tyne, the work being carried out entirely by the Company's staff under the general supervision of Mr. W. G. Townsend Gray, architect. The whole of the working drawings were prepared by Messrs. Edmond Coignet, Ltd., reinforced concrete engineers, of 125, Gower-street, London, W.C.1.

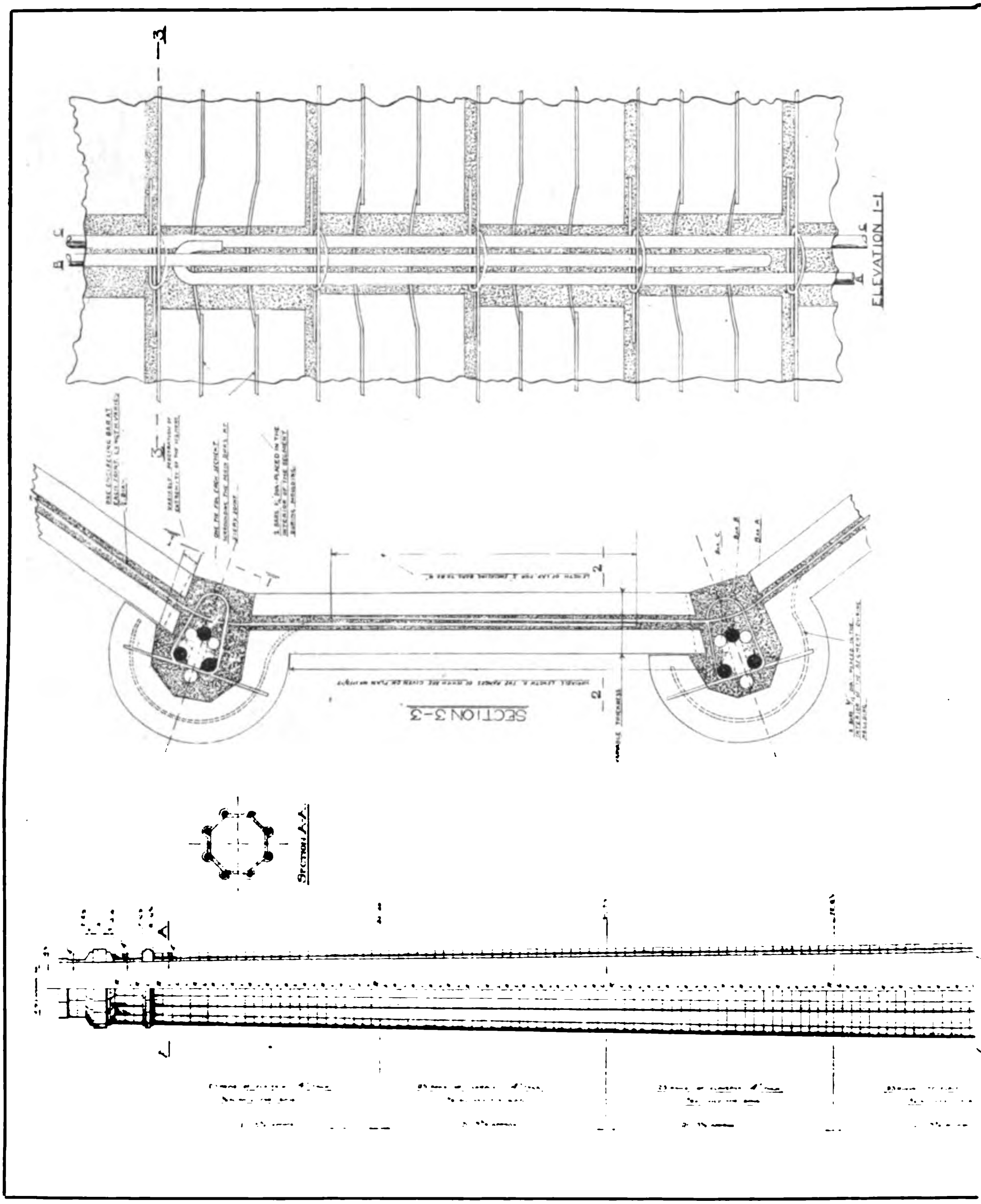


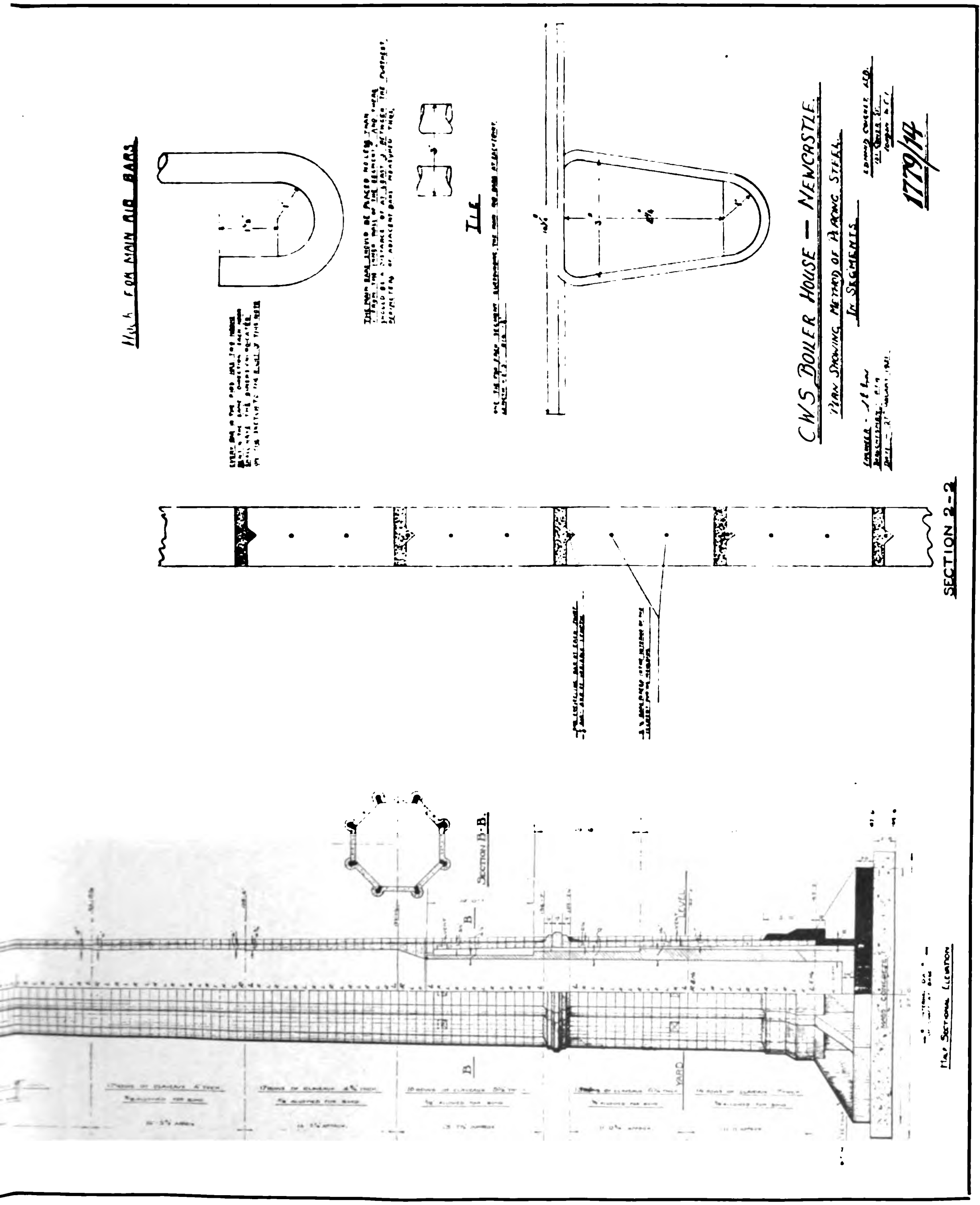
Reinforced Concrete Segmental Chimney at Newcastle.

About ten or twelve chimneys on the "Monnoyer" system had already been constructed in this country before the war. The great advantage which is claimed for this method of construction over ordinary brick chimneys is their great lightness, which has the effect of reducing very considerably the weight, and consequently the cost of the foundations required, especially where the ground is very bad. For instance, the chimney illustrated, and which is approximately 150 ft. high with an internal diameter at the top of about 5ft., weighs less than 200 tons, whereas a similar brick chimney would undoubtedly weigh about twice as much. A further advantage claimed is that apart from any economy brought about in the reduction of the weight of the foundations the cost of the shaft itself is usually appreciably less than the cost of a brick chimney.

The method of construction consists in moulding a certain number of reinforced concrete segments and bedding them on top of one another, very much on the same style as in the case of masonry work. These segments are usually about 1 ft. high by a thickness which varies according to the height. The special shape given to these segments, which have a kind of reinforced concrete hook at one end, is the principal feature of the "Monnoyer" patent.

The chimneys are constructed either hexagonal, octagonal or pentagonal, according to the diameter and height required. A pleasing appearance is produced by the fact that the various heads or hooks of each particular block form a


ELEVATION 1-1
SECTION 3-3
SECTION A-A
A
B
C
2
3



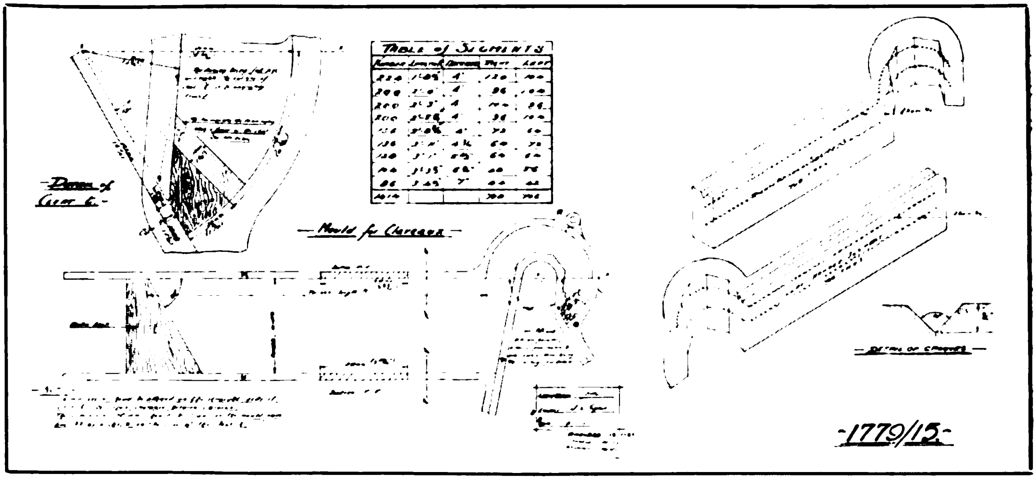
Reinforced Concrete Chimney at Newcastle-on-Tyne. (See p. 31.)

continuous longitudinal rib where the joint is formed at each corner. This rib is made for the purpose of accommodating the vertical bars which form the principal reinforcement of the chimney shaft, and which are provided in sufficient numbers to counteract any tension which may occur in the height of the chimney due to wind pressure. The tapering of the shaft is produced by gradually reducing the length of each block or segment as the work proceeds upwards.

The advantage claimed for the "Monnoyer" system over monolithic reinforced concrete chimneys is that they require no centering or scaffolding, as practically the whole of the work is prepared on the ground by pre-casting the blocks or segments in cast-iron moulds which are standardised and which can be used over and over again. For instance, in the case of this chimney only about forty moulds were used, although no less than 1,414 segments were made. Special wooden moulds were prepared for pre-casting the special blocks forming the last 10 ft. of the chimney at the top, where decorative mouldings have been introduced to enhance the appearance of the chimney. A special feature of this kind of construction is that the whole of the work can be carried out from the inside of the chimney by means of movable steel putlogs. The blocks are hoisted up by means of a pulley, and placed in position without any difficulty.

It will be noticed that the principal reinforcement, which consists of two or three round bars gradually overlapping each other within the height, is situated well away from the inside of the chimney and consequently as far as possible from the heat, whereas in monolithic chimneys the bars are usually placed circumferentially half-way through the thickness of the reinforced concrete walls where they are more liable to the effect of the heat. The segments are slightly reinforced by means of small round bars, and the thickness of each segment within the height is calculated in every case to withstand the compression due to the dead weight and the wind pressure. Each complete ring of segments is bedded on the top of the lower one by cement, a groove being left in the lower segments to accommodate a circular bar or ring, which has the effect of tying together all the segments of each ring. It will be seen, therefore, that although these chimneys are extremely light the method of construction offers a very considerable amount of strength.

The foundations of the Newcastle chimney, which are octagonal in shape with a radius of 12 ft., are composed of a reinforced concrete raft, 18 in. thick, with counterforts at each corner of the octagon. This raft is supported by a mass concrete bed 2 ft. thick. The total height of these foundations is about 7 ft. The pre-cast segments start from this level, and a rectangular opening has been left for the flues. The top and sides of



Reinforced Concrete Chimney at Newcastle-on-Tyne.

the opening are strongly reinforced as required. The total distance from the ground level to the underside of the foundations is in this case 19 ft.

In order to prevent the reinforced concrete from being affected by the heat, a brick lining has been provided of an average thickness of 14 in. for the first 20 ft., 9 in. for the next 10 ft., and 4½ in. for the following 10 ft., an airspace of a maximum of a couple of inches being provided all round the brick masonry. In order to admit the air between the brick lining and the reinforced concrete shaft, eight vent-holes were made at ground level.

NEW TYPE OF SILO.

In a new type of silo recently erected for the Kallenberg Mills, at Langensalza, the cell-walls are fitted with hollow vertical beams instead of horizontal ones. The two arched-shaped pieces which form each unit of the wall are held in place by concrete poured between adjacent pairs. Air enters the hollow parts of the walls through perforations in the latter. Only one shape and size of block is needed, so that extensive shuttering is avoided and a corresponding saving in cost and in time of construction is effected.

Fig. 1 shows a completed silo of 2,000 tons capacity; *Fig.* 2 is a sectional plan and a vertical section on the line *a-a.*—*Beton u Eisen.*

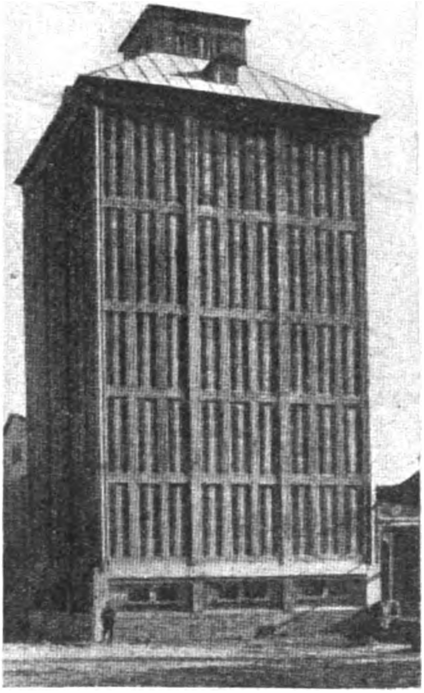


FIG. 1.—NEW TYPE OF VENTILATED SILO.

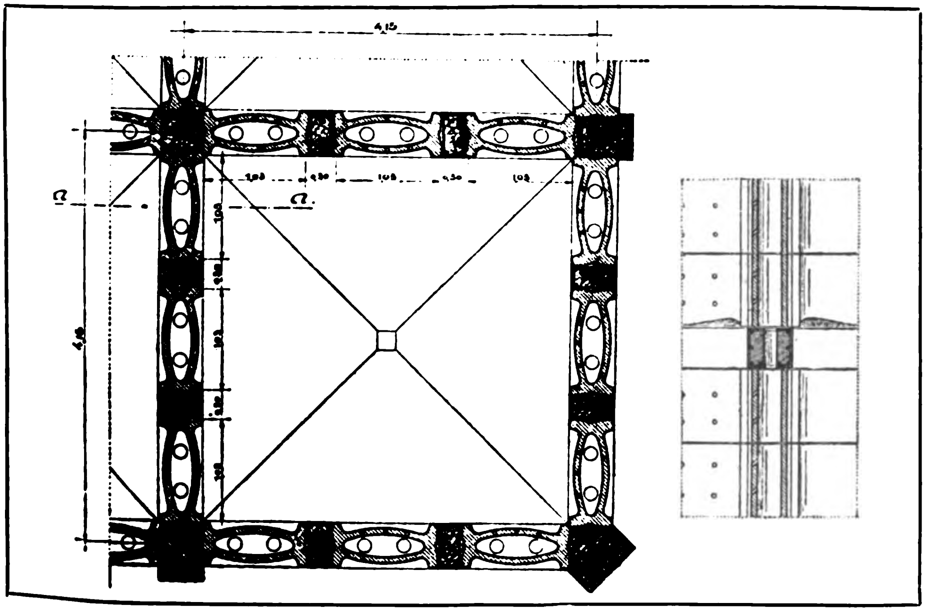
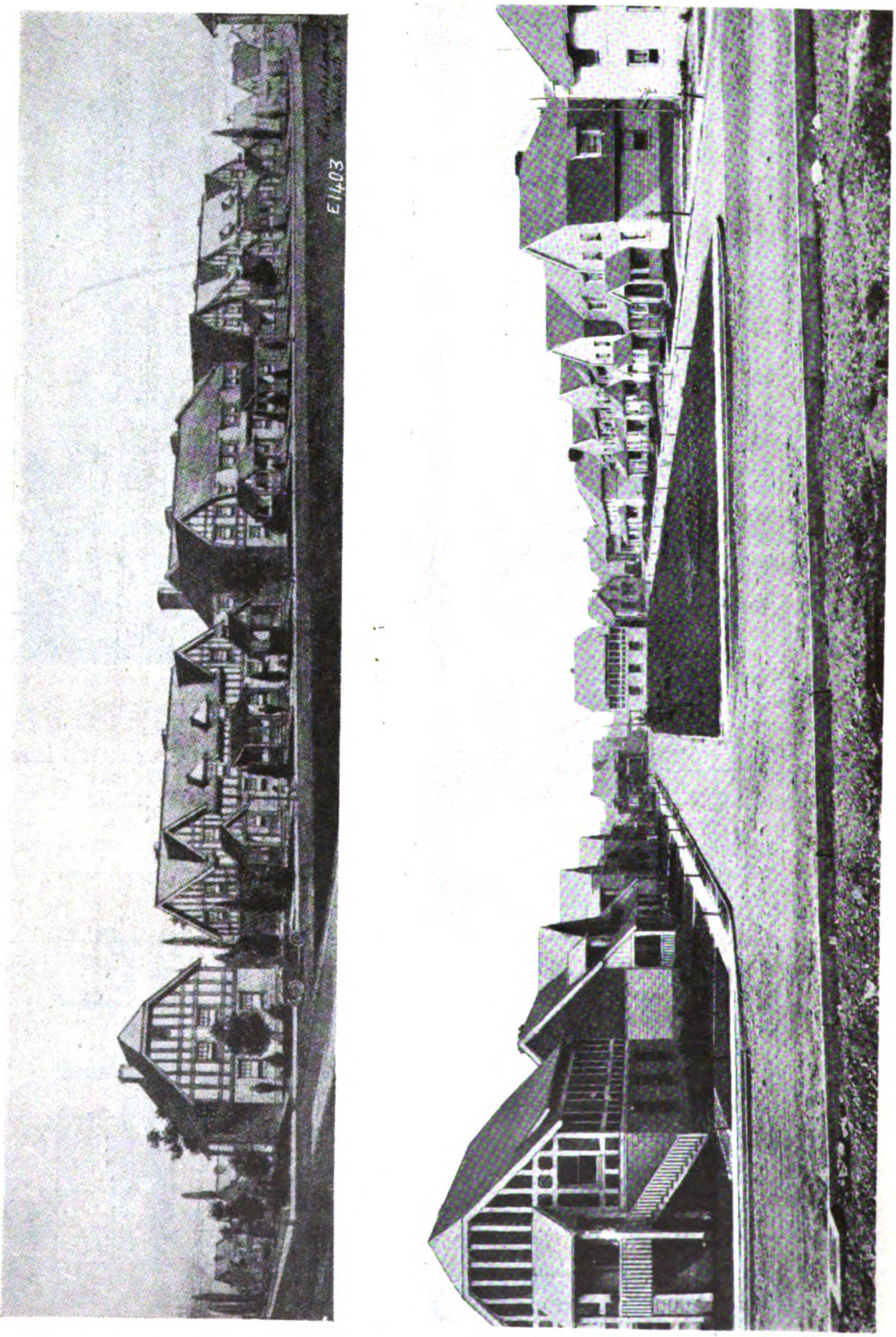


FIG. 2.—NEW TYPE OF VENTILATED SILO (*Dimensions in centimetres*).



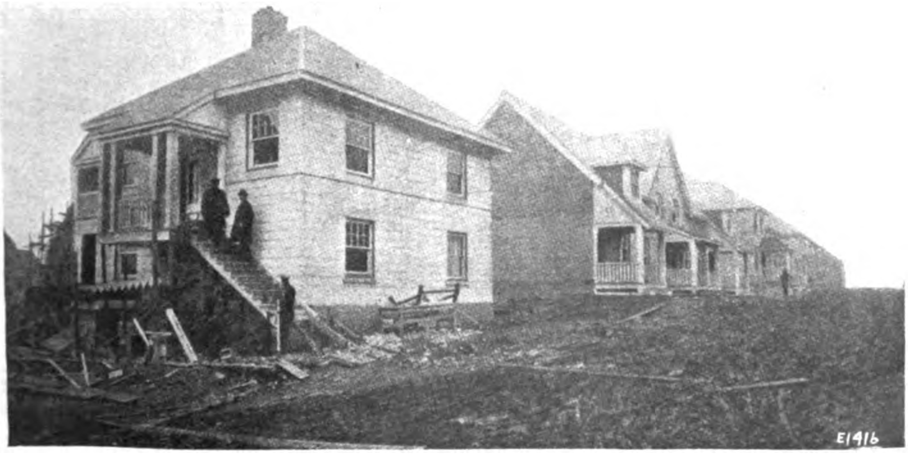
THE RE-BUILDING OF HALIFAX, NOVA SCOTIA: TYPICAL GROUPS OF CONCRETE HOUSES. (*See p.* 37.)

RE-BUILDING OF HALIFAX, N.S.

THE explosion on a ship laden with munitions, which destroyed about two square miles of the town of Halifax, N.S., occurred on the morning of December 6, 1917. Temporary houses were erected in a remarkably short space of time to accommodate the thousands who were rendered homeless, and later a reconstruction programme took definite form. The Relief Commission which had the work in hand decided on complete re-building of the devastated area, and the work has been carried out in consultation with Mr. Thomas Adams (Town-Planning Adviser to the Commission of Conservation, Ottawa), and Messrs. Ross & Macdonald, Architects, of Montreal. The reconstruction scheme is especially interesting as it involves the most extensive use of concrete units which has yet been considered in Canada.

In the new town-planning scheme it was decided to make use as far as possible of the old streets in order to take advantage of the existing water and drainage services, many of which were intact. Definite areas were set aside for residential and industrial purposes, and two new main roads, 80 ft. wide, were decided upon. Restrictions regarding the spacing of the new houses and the materials to be used were carefully considered. In the greater part of the area only the better type of buildings was permitted, with masonry, brick, concrete, or stucco walls and fire-resisting roofs. Semi-detached or terrace groups of houses had to be so constructed as to be entirely separated by fire-resisting party walls. General standards of construction, practically identical with those accepted by the United States in connection with its permanent housing programme, were fixed and rigidly adhered to.

After considering the various local materials available for use in the construction of the buildings, it was found that timber-frame construction was the most economical; followed by a form of pre-cast concrete block known as "hydro-stone"; stucco on frame; and brick, and concrete formed *in situ*, were found to be the most expensive. As a permanent and fire-resisting material was desired it was decided to make use of hydro-stone blocks, and a plant for the manufacture of this material was erected on a site where a plentiful supply of clean gravel and sand was available at low cost. The exterior and party walls of all the buildings are of this stone, and the foundations are of mass concrete; the interior partitions are finished with hand plaster. Where the concrete blocks are exposed, a faced block made of crushed granite, giving a mottled granite effect, is used. Where stucco finish is used, it is applied to the rough concrete block.

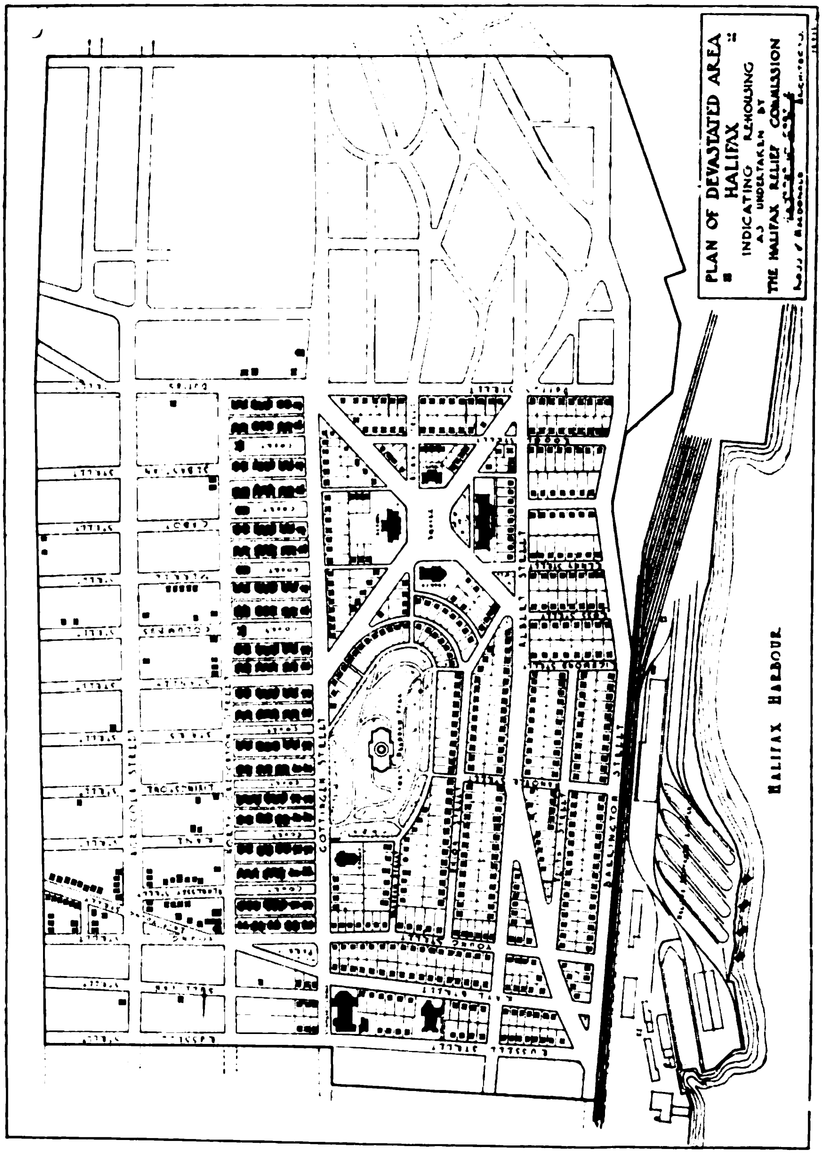


CONCRETE HOUSES AT HALIFAX, N.S.

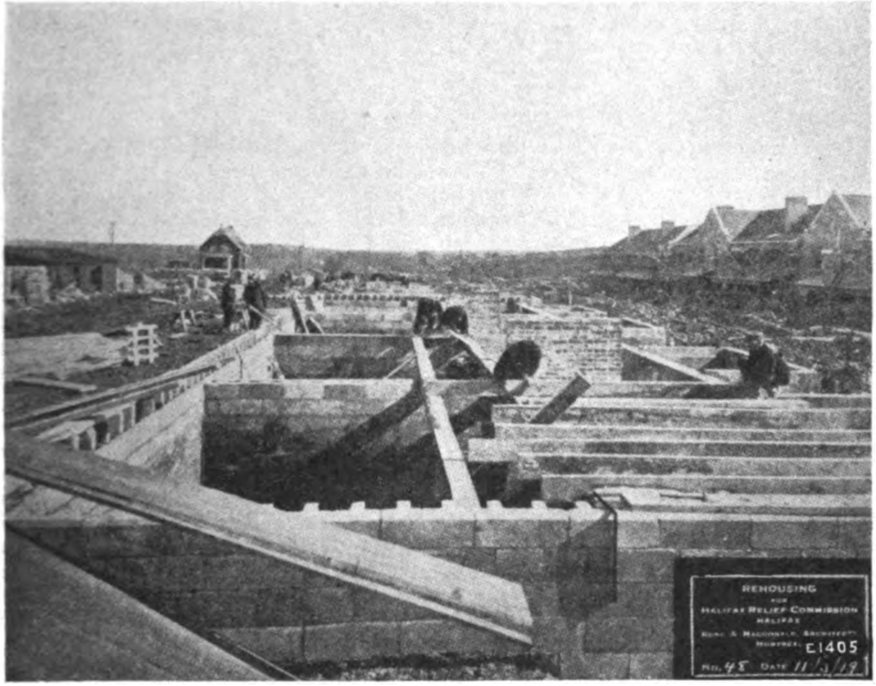
COMPARATIVE COSTS.

The cost of various methods of construction were very thoroughly investigated, and estimates for the different types were prepared for the erection of a detached two-flat house, with four rooms and bath on each floor, with outside dimensions of 28 ft. by 29 ft. in height. Since all interior finish is applicable to each type, and, therefore, costs practically the same, the consideration for comparative purposes was limited to the several forms of wall construction. Taking the lowest priced construction (No. 1) as 100, the comparative costs were determined as follows:—

No. 1.—Frame sheathing with shingle and stain, 100.


PLAN OF DEVASTATED AREA HALIFAX
INDICATING REHOUSING AS UNDERTAKEN BY
THE HALIFAX RELIEF COMMISSION
HALIFAX HARBOUR
GOTTINGEN STREET
BARRINGTON STREET

THE DEVASTATED AREA OF HALIFAX AS REPLANNED.



CONCRETE HOUSES AT HALIFAX: SHOWING CONSTRUCTION.

No. 2.—Frame sheathing with drop-siding, painted, 101·2.

No. 3.—Concrete pre-cast blocks. Exterior walls constructed of two-lug concrete blocks set in cement mortar, 103.

No. 4.—Frame sheathing, wall board, and cement stucco, 104·2.

No. 5.—Frame sheathing, metal furring, lath and cement stucco, 105·6.

No. 6.—Brick veneer, 106·9.

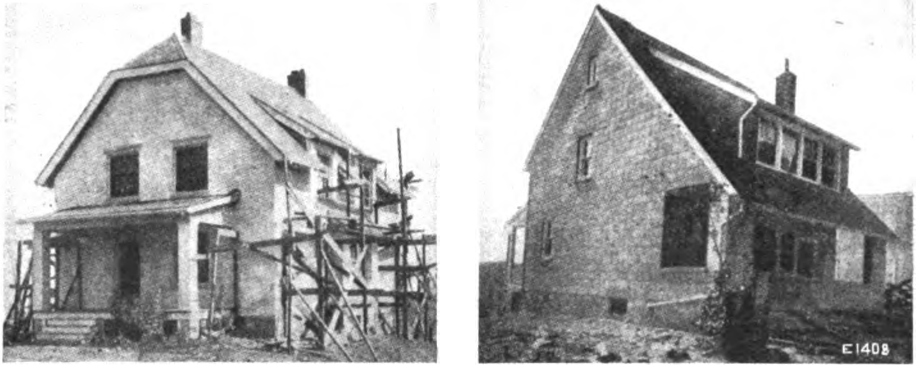
No. 7.—Solid brick, 108·8.

No. 8.—Monolithic concrete. Exterior wall poured concrete, 8 in. in thickness, rubbed to an even surface on outside face, 116.

It was found after several months of operation of the hydro-stone plant that the difference in cost between frame building with shingle (No. 1) was 5 per cent., instead of the estimate of 3 per cent., the increase being largely due to transportation costs from the works to the building site and the reduction of the estimated output of the plant owing to labour troubles.

HYDRO-STONE.

Hydro-stone, the material selected for the exterior of the houses, is a concrete consisting of gravel, crushed stone, sand and Portland cement, with sufficient water thoroughly to crystallise the cement. The mix was moulded under heavy pressure into the form of building blocks required, and then cured in steam rooms or kept moist to prevent rapid evaporation. The heavy pressure admitted of the use of a sufficiently wet mix to form a high-grade concrete, and the system of moulding provided for facing with crushed granite, marble, mica-spar or coloured sands. The blocks were made in machines estimated to exert a pressure of about 150,000 lb. against the face of the blocks. The plant contained five machines, with a total output of from 3,500 to 4,000 per day. The pressure brought to bear upon the concrete whilst in the mould squeezes out excess water and fills up the voids, forming a block with a dense face and square, sharp edges, which can be immediately removed from the mould. In the manufacture of face blocks, a white cement and white sand were used in order to obtain the full effect of the crushed granite. The facing



Concrete Houses at Halifax, N.S.

material was filled in to a depth of $\frac{3}{8}$ in. on top of the ordinary concrete, and the pressure applied at once, thus securing a thorough bond between the facing and the body of the block. When washed, the blocks were removed to a steam room, where they were cured for approximately 48 hours, and then placed in the storage yard ready for removal to the building site.

METHOD OF CONSTRUCTION.

The blocks were made 24 in. by 9 in., and were laid very quickly, a small number of masons being able to handle the output of the plant as rapidly as it could be delivered. The blocks were built up either as a cavity wall or as a solid wall. The two-piece wall comprised T-shaped units, laid in header and stretcher bond, breaking joint alternately back and front. The wall has continuous horizontal and vertical air spaces throughout; the inside and outside walls having no direct contact. Solid walls have been adopted for small buildings, with wall board on the inside of all exterior walls. The blocks have two webs projecting from the back, spaced on 12 in. centres, and are laid so as to break joint and form a strong, self-sustaining wall from 5 in. to 12 in. in thickness. Furring strips were fastened to the ends of the webs by a simple clip laid in the joint. Lathing was nailed to the furring strips and plastered.

Granite Surfaced Concrete Blocks.

Plain Concrete Blocks for Plastering.

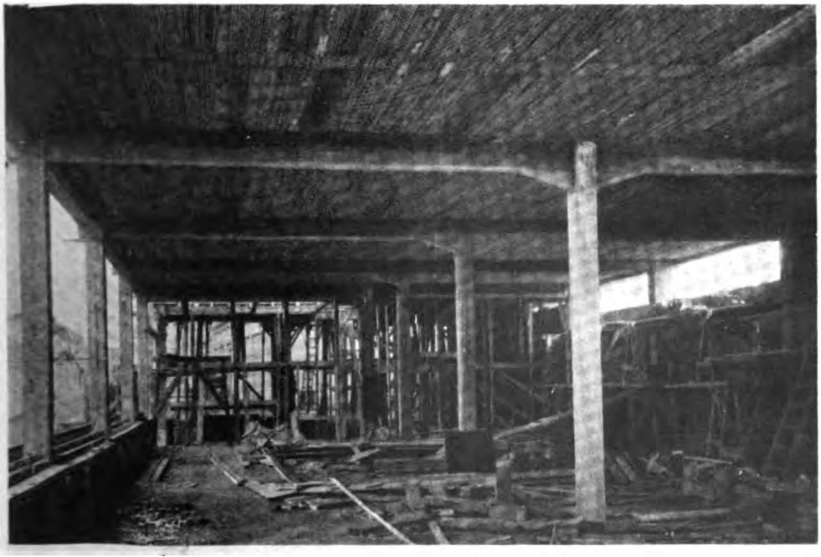
NEW METHOD OF HOLLOW FLOOR CONSTRUCTION.

A NEW type of reinforced concrete hollow floor has recently been developed which is claimed to possess many advantages over other floors of this type. It is very light, and appears to be one of the cheapest forms of floor at present on the market; it is now being used on a number of large contracts being carried out in Scotland.

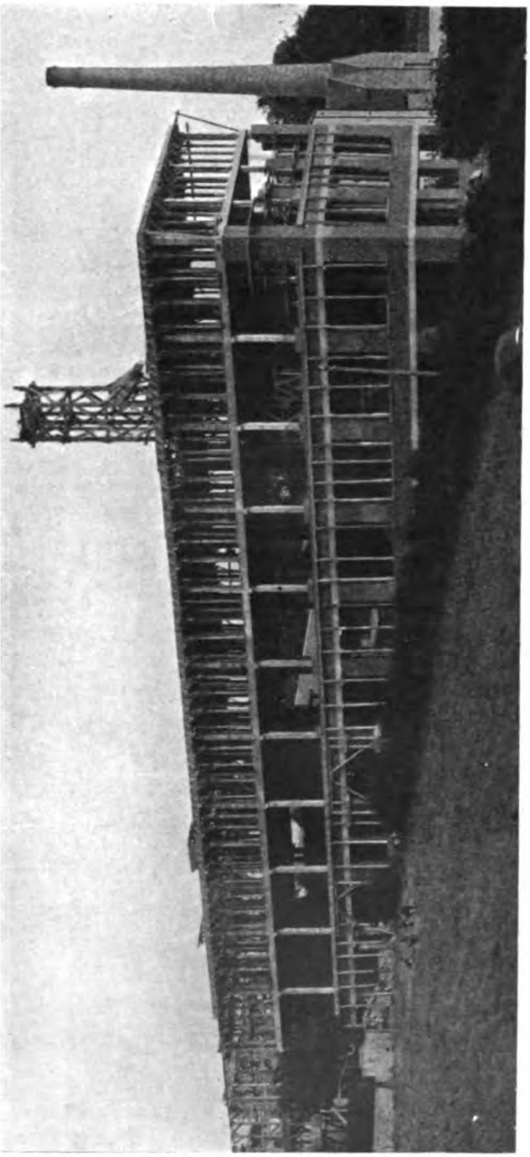
In principle it is the same as other types of hollow floors which are cast *in situ*, the essential difference being that the concrete displacers are formed of light but strong reinforced continuous wood lathing. These concrete displacers are of a rectangular section either continuous in the case of floors where the ribs span in one direction only, or they may be constructed to box form in cases where the ribs span in both directions. The lathing is cut to the required shape and is bent into the rectangular box section, the sides of the box forming the sides of the ribs, the top of the box forming the permanent centering upon which to construct the thin top slab and the bottom of the box forming an excellent surface for the reception of plaster. Ventilation may be formed by occasionally introducing small openings in the vertical ribs.

This type of floor is generally erected on the usual close-boarded shuttering employed for the tile type of hollow floor, but it has been found economical and practicable to form these floors on runners, which are introduced under each rib and which form the bottom shuttering to these members. In cases where this type of centering is employed it has been found desirable to stiffen the ribs with solid diaphragms at 3 ft. or 4 ft. apart. In this manner the lathing may be made sufficiently rigid for the men pouring the concrete to work upon it, although care must be taken by the workmen not to damage the lathing by rough treatment. After the lath troughing has been placed in position the reinforcement is introduced in the usual manner and the concrete run in.

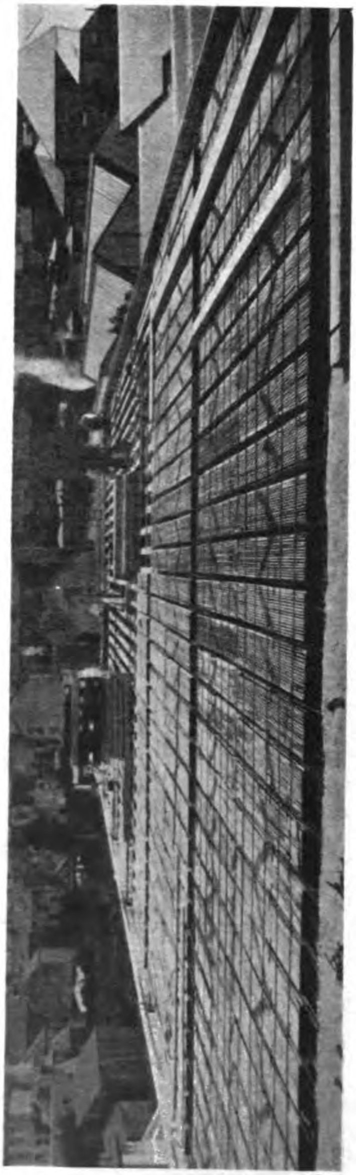
In the earlier stages of the development of this floor it was thought that the concrete might possibly become weakened by the amount of the cement mortar or liquid cement escaping through the closely-spaced lathing, but experience has proved that with concrete mixed to a proper consistency this is not the case, and that no measurable amount of cement or concrete is lost. The floor, when finished, can withstand test loads comparable with those employed for the ordinary type of reinforced concrete construction.



VIEW SHOWING UNDER-SIDE OF FLOOR.



General View.

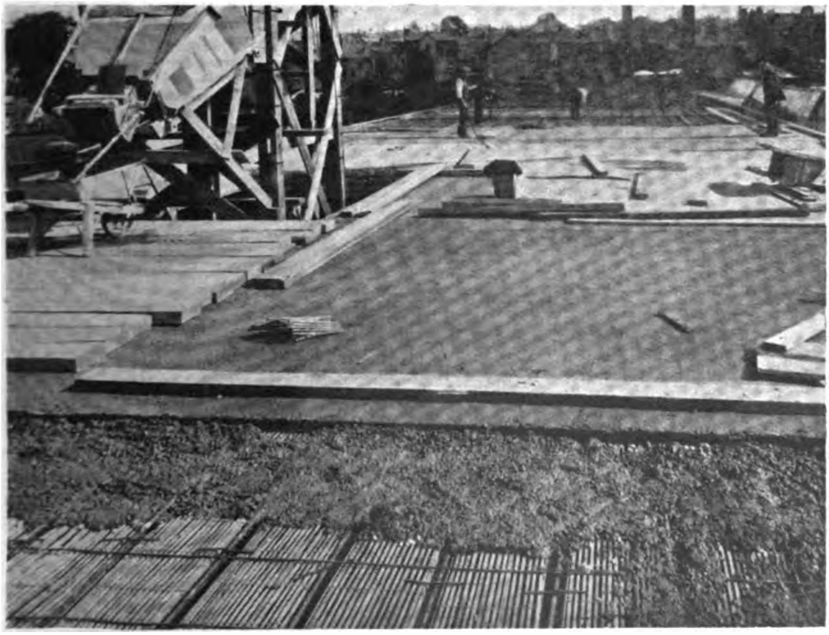


Showing Method of Floor Construction.
REINFORCED CONCRETE FACTORY BUILDING AT LANARK.

An experimental area of this floor, 18 ft. square, has been erected at the works of a leading Scottish contractor. The troughing was made, the centering erected, and the concrete poured without any more than the usual care being taken. When completed this floor was tested with a load composed of sand and a large number of workmen, so arranged as to produce bending moments far in excess of those for which it was designed, and the deflection registered during this test amounted to ⅛ in., or 1/1728 of the span. On this satisfactory test the floor was adopted in a large hosiery factory erected at Lanark for Messrs. McDougall. This factory is now completed, and the total area of this type of flooring laid amounts to nearly 3,000 sq. yds., and carries various superloads up to 1½ cwts. per sq. ft. over clear spans of more than 15 ft. The photographs reproduced herewith show the floor in its various stages. The floors to this particular job have been constructed with the ribs running in one direction only, but where the overall depth of the floor is an important factor ribs spanning in both directions may be introduced with advantage.

The economy resultant from the adoption of this type of floor in a building containing many stories can be seen when it is stated that in a floor spanning 20 ft., and carrying a load of 1 cwt. per sq. ft., the overall thickness of the flooring amounts to 10 in., thereby ensuring a rigid sound-proof floor, weighing only 40 lb. per sq. ft. This figure is, of course, exclusive of the weight of the plaster forming the ceiling and any floor surfacing that may be required by the architect. With so light a floor the design of the supporting beams and columns can be made much lighter, and the economy resulting from this in a large job becomes very considerable. With regard to the cost of the hollow floor itself, the elimination of the necessity for close-boarded shuttering, the small cost of purchasing and making the concrete displacers, and the economy in steel reinforcement due to their adoption, are factors which permit of considerable savings being effected.

Further particulars regarding this type of reinforcement may be obtained from Messrs. The Weston-Reuter Co., Ltd., of No. 2, Dean Farrar Street, S.W.1. A patent has been applied for.



NEW METHOD OF HOLLOW FLOOR CONSTRUCTION: WOOD LATHING IN FOREGROUND.

REINFORCED CONCRETE TELEGRAPH POSTS.

Our issue of October last contained a description of a telegraph post designed on a new principle by Messrs. F. W. Bradshaw, engineers, of Hitchin, and named the " Boom " Type Post. A trial test at thirty days gave very promising results, which were more than borne out by the test at ninety days which was recently made at the Arlesey depot of Messrs. Tidnams, of Wisbech, and of which the following is the official report :

Subject.—One " Light " line pattern reinforced concrete telegraph post, " Boom " type, after ninety days' maturing : Length over-all, 30 ft. ; length out of ground, 25 ft. ; length below ground, 5 ft. Designed for a 6 cwt. crippling load, or with a factor of safety of 3 = 2 cwt. working load. (The term crippling load being understood to mean one which would extend to the limit of elasticity and produce a permanent " set.")

Weight.—At thirty days the post weighed 6 cwt. 1 qr. At the end of maturing this weight would probably be reduced to 6 cwts.

Details of Test.—A post was placed in the ground and its foot inserted within a concrete casting made in halves and bolted together. The spaces between the outside of this casting and the sides of the hole in the ground were packed tightly with bricks. The test rope was applied and made fast at a point 23 ft. above ground-level and carried horizontally over a pulley fixed to another telegraph post erected about 35 ft. in front of and away from the post to be tested. The post was tested at right angles to the line of overhead wires, i.e., in the direction of greatest wind stress, with the following results :—

Weight Applied. Cwt.	Deflection. Inches.
1	¾
1¼	1½
1½	2
1¾	2¾
2	3½
2¼	4
2½	5
2¾	5½
3	6¼
3¼	7¼
3½	8¼
3¾	9
4	10
4¼	10¾
4½	11¾
4¾	12¼
5	13½
5¼	14¼
5½	15¾
5¾	16½
6	17¼
6½	19¼
7	20½
7½	22¼
8½	26

At 9¾ cwt. the coupling bolts of the foot casting snapped and the ground gave way by reason of the leverage upon it. Post gave no signs of breaking.

At 2½ cwt. a slight movement of the post at ground-line appeared.

At 4½ cwt. the bodily movement of post in the earth was ⅛ in. at ground-line ; at 5½ cwt. the movement appeared to be 3/16 in. at ground-line.

At 6 cwt. (i.e., the expected breaking-point) no permanent set had taken place, but very faint hair cracks became visible.

The loads were then gradually released and the deflections noted were as below :—

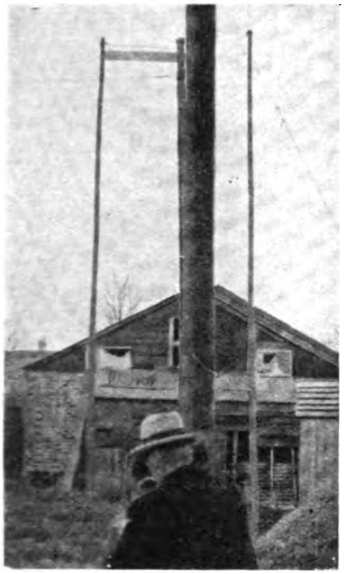
Weight Cwt.	Deflection. Inches.
6	17¼
5	17¼
4	16¾
3	16½*
2	11
1	5½
Zero.	1†

The weights were again applied up to 9¾ cwt., at which point the ground gave way, but no set or crack appeared in the post, and on releasing the load the post resumed practically its normal condition.

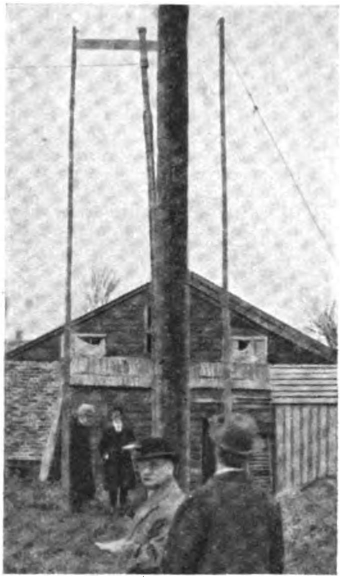
We illustrate on the following page photographs of different stages of the test.

* At this point all hair cracks disappeared.

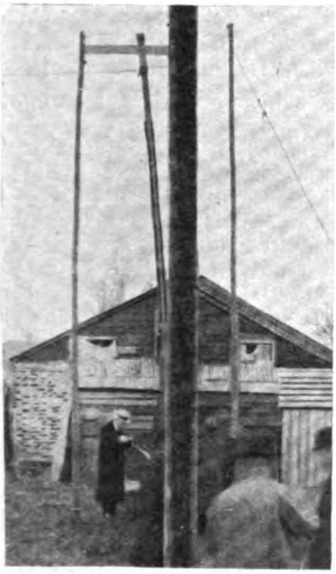
† Difference due to ground movement. Post absolutely true and straight.



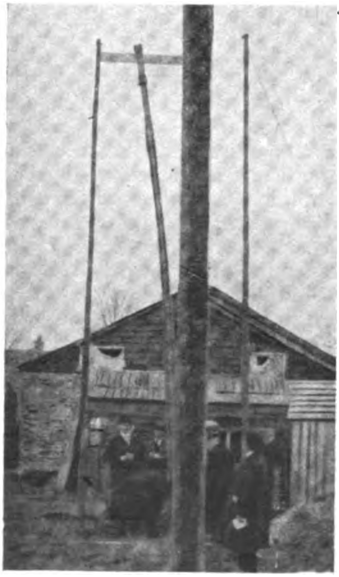
Showing Deflection at 2 cwt. Load.



Showing Deflection at 4 cwt. Load.



Showing Deflection at 6 cwt. Load.



Showing Deflection at 9½ cwt. Load.

[The Wooden Pole in the foreground is the guide-pole.]

TESTS ON A REINFORCED CONCRETE TELEGRAPH POST. (*See p.* 44.)

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—ED.

THE EFFECT OF HYDRATED LIME ON CONCRETE.

QUESTION.—Would you give me an opinion as to the effect of the addition of hydrated lime to the extent of 10 to 15 per cent. of the volume of cement in concrete ? What is its effect as a filler or waterproofing agent ? Has it a detrimental effect on the strength, either tension or compression ?—E. H. B.

ANSWER.—The effect of additions of hydrated lime to concrete has been the subject of much discussion in America, and after a series of exhaustive tests the following conclusions have been arrived at by Professor Duff A. Abrams of the Structural Materials Research Laboratory, Chicago.

(1) Hydrated lime had little effect on the absorption of dry concrete.

(2) Each 1 per cent. of hydrated lime (in terms of the volume of cement) reduced the compressive strength 0·5 per cent.

Applying these conclusions to the foregoing questions, it will be seen that as a filler or waterproofing agent hydrated lime is ineffective, otherwise it would have decreased the absorption of dry concrete.

The addition of hydrated lime to the extent of 10 to 15 per cent. of the volume of cement would reduce the strength of the concrete by 5 to 7½ per cent.

REINFORCED CONCRETE GLAZING BARS.

QUESTION.—I am proposing to use reinforced concrete glazing bars in the lights in a reinforced concrete flat roof, and should be extremely obliged to have your opinion on the possibility of using something cheaper and less elaborate than what seems to be on the market. I should prefer to make the bars myself, and of a more simple form without any expensive copper or brass screws and fittings. However, I do not know if there is any objection to copying the practice with timber, even as far as using putty, unless it may be that the putty is perishable. Of course, cement will hold down the glass, but in this particular instance it may be subject to frequent renewals for breakages, and would no doubt be so effective as to be a trouble to replace. Can you suggest anything else more suitable ?—R. F. B. T.

ANSWER.—Putty glazing has the well-known defects of letting water through when the putty has ceased to be plastic, and is cracked by expansion and contraction. In addition, the linseed oil in it will be saponified by the alkalies of concrete. Unless the writer is prepared to experiment with much expenditure of time and money he would probably do better to use something which has been tried out.

REINFORCED CONCRETE FLOORS.

QUESTIONS.—(1) Does the cement coat on reinforced concrete floors require any special treatment at the beginning or from time to time to keep it in good condition and to avoid dryness and dust when it is made with good Portland cement ?

(2) Which are the best jointless plastic finishes for reinforced concrete floors in the various parts of a country house, including the stairs ? What is its usual thickness ?

(3) Can the same finish be employed

for the nosings of the steps, when treads and risers are finished either with cement coat or with jointless plastic finish ? Is it not necessary to protect the nosings ?

(4) How is the so-styled "polished cement, enamelled" prepared ?—T. M. V.

ANSWERS.—(1) This depends on the use to which it is put. If the surface is finished with good 3 to 1 granolithic or equal, and kept moist for fourteen days in a warm atmosphere, it does not dust under ordinary wear. A good soaking with silicate of soda reduces the dusting tendency greatly.

(2) One of the best jointless finishes is the so-called "patent" flooring made from magnesite, asbestos, and fillers, of which "Decolite," made by Bells United Asbestos Co., Ltd., is a good example. Care must, however, be taken that it is laid fairly dry, and on no account must the floor be swamped with magnesium chloride solution, as risk of corrosion of the reinforcement would then occur. The best safeguard is to employ only a well-known firm of repute under guarantee. The usual thickness is ½ in. and the floor should be left rough.

(3) Yes. Need for nosing protection depends entirely on use. For country houses no protection should be necessary.

(4) We have no information on this subject.

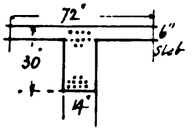
SOUND-PROOF FLOORS.

QUESTION.—I should be glad of a reply to the following :—It is proposed to construct the floors of a building (to be used as firemen's living quarters) in reinforced concrete. The building will be about five stories high. Can you advise as to the most sound-proof methods of construction for floors and walls, at a reasonable cost ? In a similar building now in use having the ordinary 3 in. to 4 in. thick slab and beam reinforced concrete floors and brick partition walls, complaints are made by the occupants of neighbouring flats of hearing too much of one another.—T. P. G.

ANSWER. — The most sound-proof method of constructing the walls, consistent with the terms of the question, would be to make the building a frame building, with panel walls of two thicknesses 4½ in. thick with a 2-in. air-space between. The floors should be hollow tile and reinforced concrete, with lime and hair plaster under (*not* a hard plaster), and, if possible, rubber floor finish stuck down continuously.

REINFORCED CONCRETE BEAMS.

QUESTION.—How can the moment of resistance of a T beam having compression reinforcement be calculated ? As an example I should like to have the following case worked out, the limits of stress being 600 and 16,000 lb./sq. in. : Tension and compression reinforcement = No. 10 1¼ in. rods.



Would the reinforcement be better arranged in two layers each of five bars ?

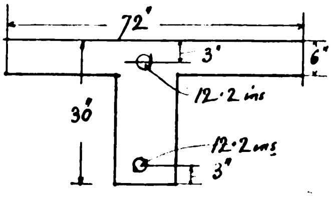
I should also like to know the best way of arranging the tension reinforcement, recommended in a book I have, for certain beams.

(i) 5 1 in. and 5 1¼ in. bars in a beam 12 in. wide.

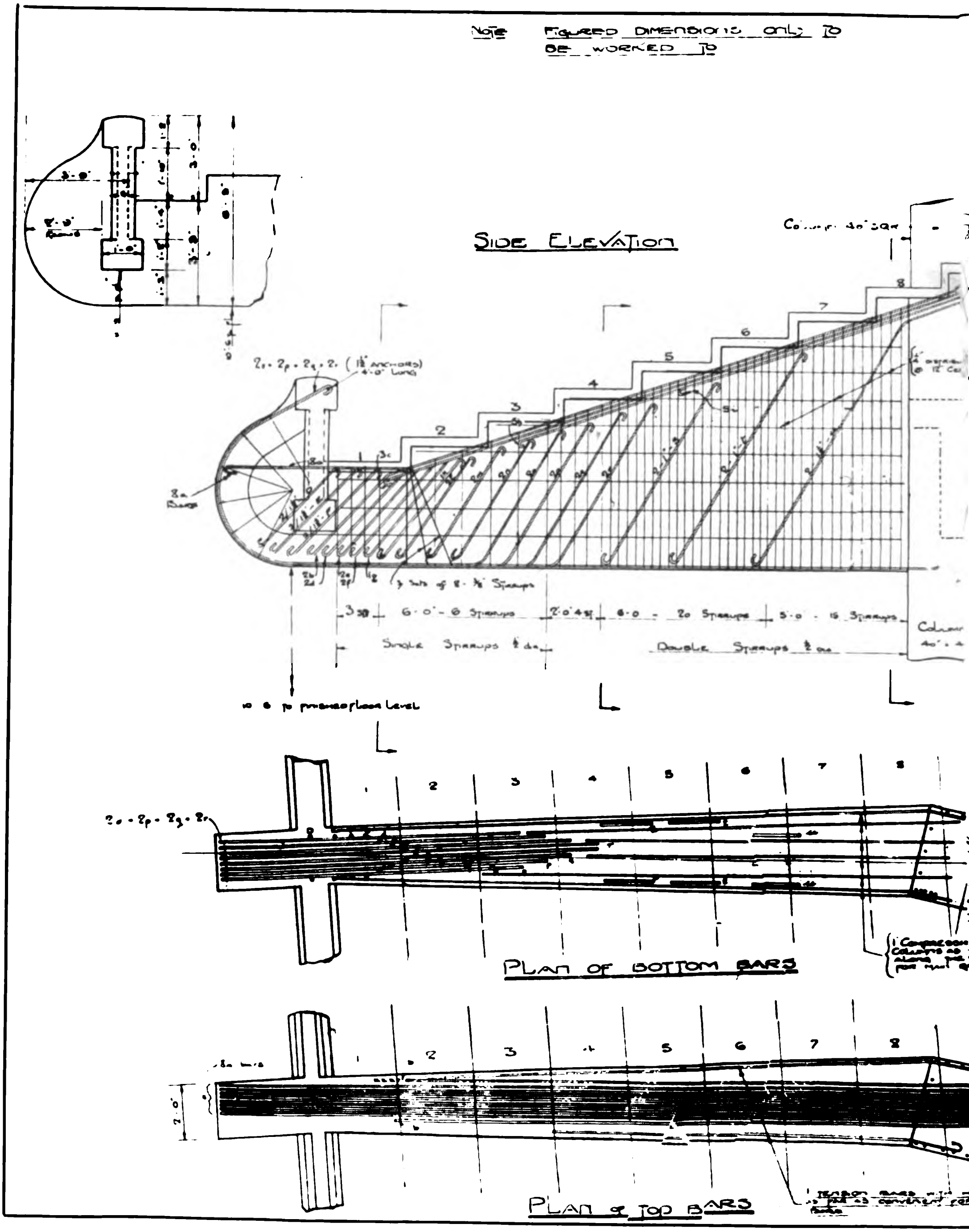
(ii) 3 1 in. and 5 1⅛ in. bars in a beam 10 in. wide.

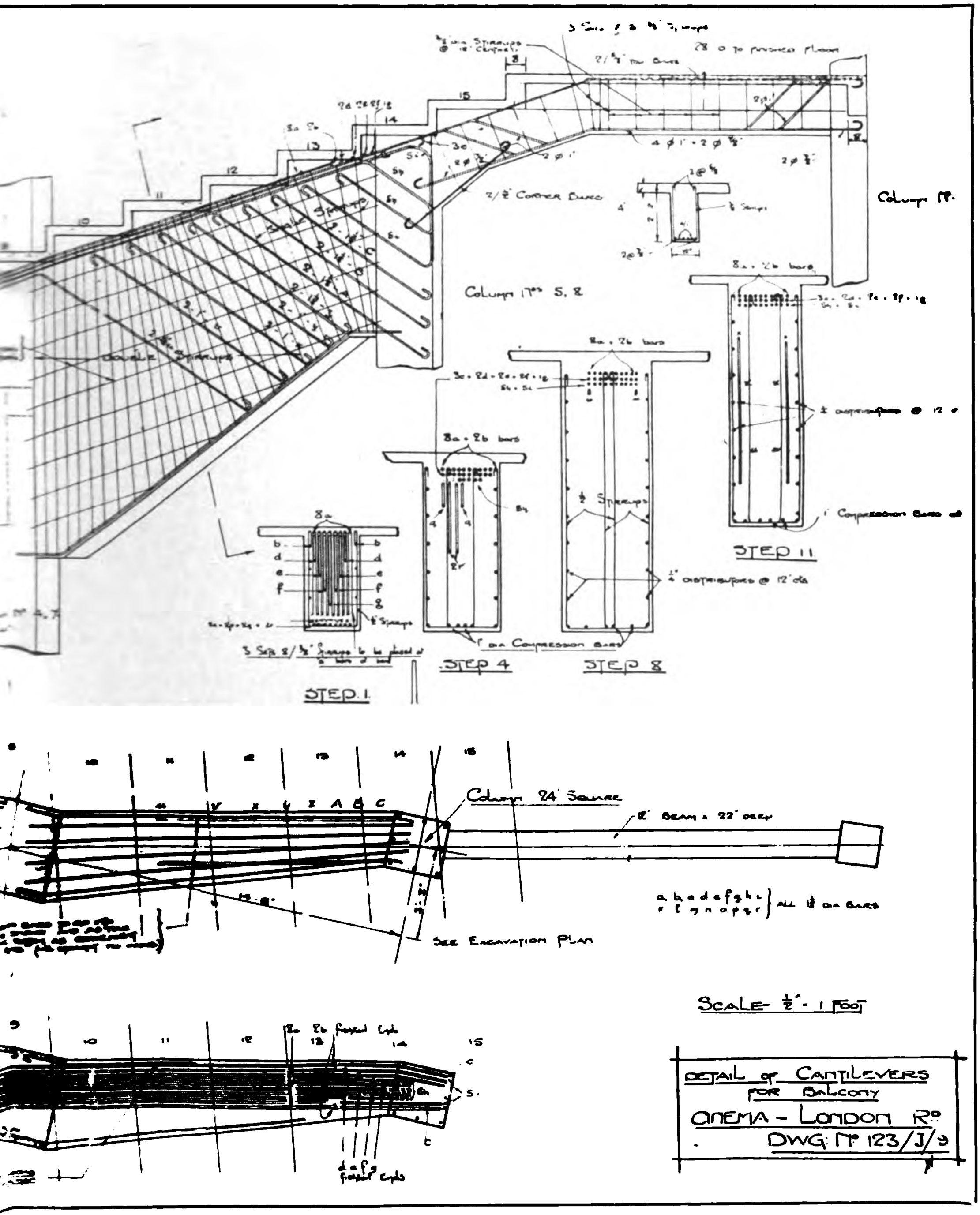
These beams do not seem to me wide enough for a single layer of five rods, but yet I presume the rods should be placed symmetrically.—A. G.

ANSWER.—We will assume the centre of gravity of the steel to be 3 in. from the upper and lower surfaces respectively.



[*Continued on p.* 50.


Note Figured dimensions only to be worked to
Side Elevation
Column 40" sq
2o · 2p · 2q · 2r (1½" anchors) 4'·0" long
8a Bars
8a
3c
1
2
3
4
5
6
7
8
2b 2d
2e 2f
1g
3 sets of 8 · ⅜" Stirrups
3'3" 6·0" – 6 Stirrups
2'0" 4 st 6·0 – 20 Stirrups
5·0 – 15 Stirrups
Single Stirrups ½ dia.
Double Stirrups ½ dia.
Column 40" × 4
10'6" to finished floor level
Plan of bottom bars
2o · 2p · 2q · 2r
Plan of top bars
8a bars
2'0"



…inema at Portsmouth. (*See p.* 15.)

Continued from p. 47.]

Then the percentage of tension steel on the equivalent rectangular beam (see Faber & Bowie's "Reinforced Concrete Design," Vol. I, p. 39) is

$$p = \frac{12{\cdot}2 \times 100}{72 \text{ in.} \times 27 \text{ in.}} = 0{\cdot}63 \text{ per cent.}$$

The moment of resistance at usual stresses would be

$$R = 12{\cdot}2 \times 16{,}000 \times 0{\cdot}88 \times 27 = 4{,}600{,}000 \text{ in. lb.}$$

Depth of neutral axis will be

$$0{\cdot}36 \times 27 \text{ in.} = 9{\cdot}7 \text{ in.}$$

The total compression force will be :—

$$\frac{4{,}600{,}000}{{\cdot}88 \times 27} = 193{,}000 \text{ lb.}$$

The average stress in the concrete in the slab, with 600 lb./in. at the top, will be

$$600 \times \frac{9{\cdot}7 - \frac{6}{2}\text{in.}}{9{\cdot}7} = 415 \text{ lb./in.}$$

Hence compression carried by concrete in slab =

72 in. × 6 in. × 415 = 179,000
Extra compression carried by steel
12·2 × 14 × 415 = 71,000
250,000 lb.

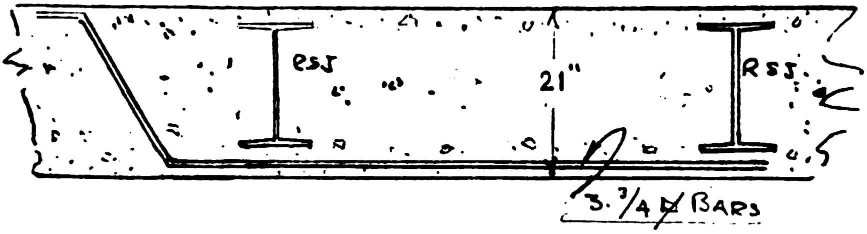
This shows that the compression flange is stronger than the tension flange, which therefore limits the strength. The compression steel is much more than is required. In fact, the concrete would not be much overstressed if it were omitted.

The arrangement of reinforcement depends on many things, such as the bending up arrangement for shear, lapping with the next beam, etc., and we cannot say what would be best with the information given. Certainly an unsymmetrical arrangement like 3 1 in. and 5 1⅛ in. is bad practice.

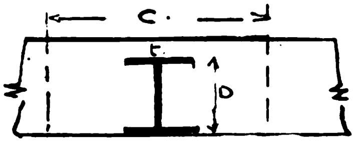
REINFORCED CONCRETE CONSTRUCTION.

QUESTION.—I should be obliged if you could deal with the following two points :

(1) A case has recently occurred in which it was required to support the ends of two existing R.S.J.s carrying an upper floor on a new reinforced concrete beam. The B.M. was calculated in the usual manner, and the size of the new beam worked out at 21 in. by 10 in. with 3¾ in. square bars as tensile reinforcement. The question then occurred, must the new beam be formed completely under the ends of the R.S.J.s, or could headroom be gained by allowing the ends of the joists to come down into it ; if so, how close to the upper surface of the tensile reinforcement could they be safely kept, and what supplementary stresses would be set up by this arrangement.



(2) What is the most approved formula for calculating the increased M.R. of R.S.J.s embedded in solid concrete floors or bridge decks ? A formula given by Messrs. Redpath, Brown & Co., Ltd., in their Handbook gives varying results in accordance with the value assumed for C.—viz., the width of concrete considered to be acting with the joist. Should this be taken as the spacing of the joists C. to C. or should the rules for finding "*b*" when calculating **T** beams be applied to this case also.—H. M. P.



ANSWER.—(1) If the concrete between the joists is solid and good, the beam will be as strong as if they were not there. There is no harm in the bottom of the joists touching the reinforcing rods.

(2) Redpath Brown's formula may be followed. The distance C should be as for **T** beams, but in nearly all cases this will coincide with the spacing centre to centre of joists.

THE INSTITUTION OF STRUCTURAL ENGINEERS.

RECTANGULAR SLABS.

At the last meeting of the Institution of Structural Engineers, held at No. 296, Vauxhall Bridge Road, S.W 1, on December 14, Mr. A. Ingerslev * read a paper on "The Strength of Rectangular Slabs," which, he said, was limited to such slabs as had an appreciable thickness compared with the span, and more particularly to slabs of reinforced concrete only. The following is an abstract of the paper :—

The theoretical solution of this problem, as regards the determination of the bending moment and stresses at any point of the slab, has been approached by several scientists for more than a century. But a theoretical solution, however complete, does not solve the whole problem, as the cause of failure is not exactly known. Finally, a theoretical solution cannot be more accurate than the assumptions from which it starts, and these assumptions in the case of reinforced concrete slabs are known to be incorrect inasmuch as rectilinear distributions of stresses are assumed, and in most cases it is assumed that Poisson's ratio is equal to zero.

It is proposed to indicate how a solution of the problem may be found by elementary calculus. The aim should be to indicate a formula by the use of which rectangular slabs would be so designed as regards thickness and reinforcement as to offer the same factor of safety against failure as other members of reinforced concrete structures. It then becomes only a matter of choosing the correct factor of safety to ensure against failure. The factor of safety here referred to may be defined by the equation :

$$fs = \text{Ultimate Load}/\text{Working Load}$$

where the working load includes the dead weight and the superimposed weight. It should be noted that this does not necessarily mean that subject to the working load the stresses are $1/fs$ of the ultimate stresses of the materials ; nor does it necessarily mean that the bending moment at any point of the slab does not rise above the value of $1/fs$ bending moment at failure. But the definition as it stands enables us to approach the problem of studying the conditions of the slab immediately before failure, and these, in the case of reinforced concrete slabs, are far less complicated at failure than at the stages before.

A slab immediately before it breaks appears to be divided up by cracks in four portions (*Fig.* 1). Although the cracks appear first near the centre, at failure they are evenly distributed right from the centre to the corners of the slab. This would indicate that the stresses have been equalised after the steel has passed the elastic limit, when deformation of the slab takes place without corresponding increase in stress.

A reasonable assumption, therefore, is that the bending moment is uniformly distributed across the main lines along which the rupture takes place. Experiment and experience show that this assumption cannot be far wrong, and it offers an elementary solution of the problem. Each of the four portions of the slab, if cut away, should remain in equilibrium, subject to the forces acting upon it. There is no shearing force at those sections where the bending moment is a maximum. Therefore the forces to be considered are only : (1) the total working loads on the portion of the slab, (2) the upward reaction along the edge forming the support, and (3) the bending moments along the cut edges.

It is known that the bending moments are vector quantities, subject to the laws of vector addition just as much as forces and other directed quantities are. If the bending moments along the lines of rupture are represented by vectors, each of these, from the assumption that the bending moment is uniformly distributed along the lines of rupture, might, by choosing a suitable scale, be represented by a line having the length of the corresponding edge. The sum of these vectors then for each portion of slab would be exactly the length of the supporting edge.

Let B = *Bending* moment per unit length.
W = Total *weight* on slab.
A = *Area* of slab.

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* Mr. Ingerslev is a partner in the firm of Peter Lind & Co., of Westminster.

w = *Weight* per Area = $\frac{W}{A}$.

Ab = *Area* of the portion of the slab contiguous to b.

Al = *Area* of the portion of the slab contiguous to the length l.

db = *Distance* of the centre of gravity of the Area Ab from the side b.

dl = *Distance* of the centre of gravity Al from the side having the length l.

h = The height of the triangle having the base b.

The following conditions must apply:

$$B=\frac{1}{6}\cdot\frac{wbh^2}{b}=\frac{1}{4}wb^2\,\frac{\left(\frac{l}{2}-\frac{2h}{3}\right)}{1}.$$

$$\therefore\ h=\frac{1}{2}b\left[\sqrt{3+\left(\frac{b}{l}\right)^2}-\frac{b}{l}\right]$$

The intensity of the bending moment (i.e., bending moment/width of cross-section) is given by the equation:

$$B=\tfrac{1}{6}\,wh^2.$$

In the case of a square slab this formula gives $B=\frac{1}{24}\,wl^2$ and in the case of a very long narrow slab for which $b/l=0$ it gives $h=\frac{b}{2}\cdot\sqrt{3}$ and therefore $B=\frac{1}{8}wb^2$. This formula further indicates that our fundamental assumption of uniformly distributed bending moment along the lines of rupture leads to the angle of the lines radiating from the corners changing from 45° in the case of a square slab to 60° in the case of a long narrow panel. This is also confirmed by the appearance of slabs actually tested (*Fig.* 1). If the bending moment be expressed by the total load on the panel, the formula becomes

$$B=\frac{W}{24}\cdot\frac{b}{l}\left[\sqrt{3+\left(\frac{b}{l}\right)^2}-\frac{b}{l}\right]^2$$

The results of the formulas are set forth in the Table on next page.

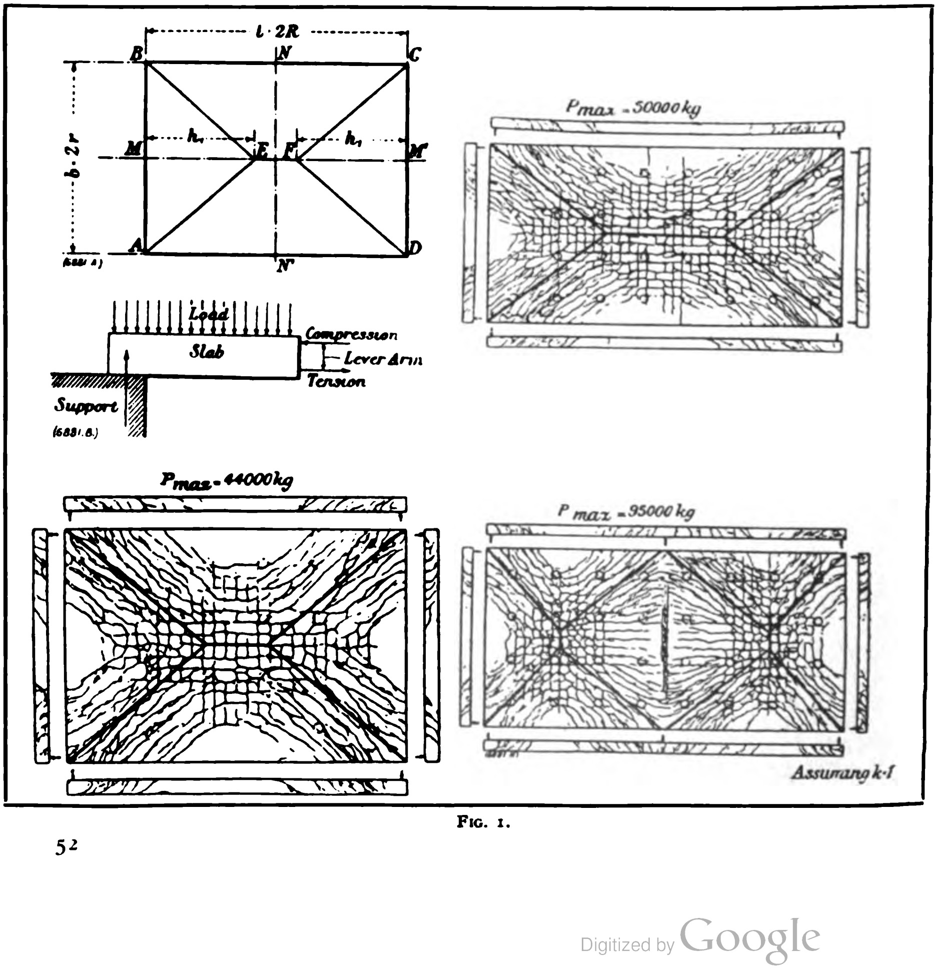


FIG. 1.

			B.M. in Slab.		Reactions on Supports.		If uniformly distributed.	
l/b	$2h/b$	L	wb^2/B	W/B	W/Rb	W/Rl	W/Rb	W/Rl
		° ″						
1·0	1·000	45·00	24·00	24·00	4·00	4·00	4·00	4·00
1·1	1·047	46·20	21·95	24·07	4·20	3·82	4·20	3·82
1·2	1·089	47·30	20·25	24·30	4·42	3·66	4·40	3·66
1·3	1·126	48·25	18·95	24·63	4·62	3·53	4·60	3·54
1·4	1·159	49·10	17·85	25·00	4·84	3·42	4·80	3·44
1·5	1·189	49·55	16·95	25·45	5·05	3·32	5·00	3·34
1·6	1·216	50·35	16·25	25·94	5·27	3·22	5·20	3·26
1·7	1·241	51·10	15·60	26·50	5·46	3·16	5·40	3·18
1·8	1·263	51·40	15·05	27·05	5·70	3·08	5·60	3·12
1·9	1·283	52·05	14·60	27·65	5·91	3·02	5·80	3·06
2·0	1·303	52·30	14·14	28·28	6·14	2·97	6·00	3·00
2·5	1·378	54·00	12·62	31·55	7·26	2·76	7·00	2·80
3·0	1·431	55·00	11·70	35·10	8·39	2·62	8·00	2·67
∞	1·732	60·00	8·00	—	$wb^2/2 \cdot 3$	2·00	—	—

It will now be shown how the load on the slab is distributed on the four edges. There is no shearing force along the main lines of rupture; the total load on each supporting edge, therefore, is the same as the load on the portion of the slab contiguous to the support. Furthermore, the load must be distributed along each edge approximately in accordance with the shape of these figures; that is, the load is either triangular or trapezoidal.

On this assumption the bending moment per width of cross-section along the lines MM′ and NN′ (*Fig.* 1) would be calculated as follows: The areas of the triangle and trapezoid being Ab and Al respectively and the distance from their centre of gravity to the respective sides being db and dl the bending moment along MM′ is, therefore:—

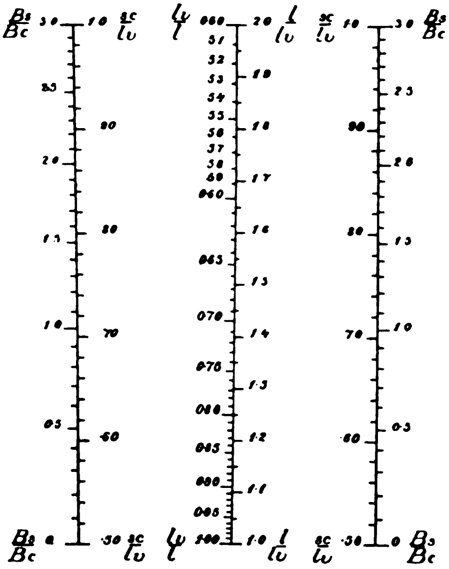
$$Bl = w \,.\, Al \,.\, b/2 - w \,.\, Al \,.\, (b/2 - dl) =$$
$$w \,.\, Al \,.\, dl = \tfrac{1}{6}\, wh^2$$

and along NN′ it is

$$Bb = w \,.\, Ab \,.\, l/2 - w \,.\, Ab\,(l/2 - db)$$
$$= w \,.\, Ab \,.\, db = \tfrac{1}{6}\, wh^2.$$

As the intensity of the bending moment along NN′ would not exceed that along the main lines of rupture, the load on the supporting edges is probably gathered even more towards the centre than is indicated by these lines. It is interesting to compare this result with what would obtain assuming that the load was uniformly distributed along the periphery of the slab (*see table*). It would appear that the load on the panel, according to the present assumption, is very nearly borne by the supporting edges in proportion to their length.

The problem of the slab that is fixed along one or more of the supporting edges may be approached in similar manner. It should be noted, however, that it is not correct to assume an even distribution of the bending moment over the supporting edge. The results, however, will not be affected if the bending moment introduced in the formula is the *average* of the bending moment in the slab along the edge. Assuming that the bending moment at the centre is B, the average bending moment per length of supports may be written as $B_1 = C \,.\, B$; $B_2 = C_2 \,.\, B$, $B_3 = C_3 \,.\, B$, and $B_4 = C_4 \,.\, B$ respectively.



Bs — *Bending Moment across Support.*
Bc — *Bending Moment at Centre.*
sc — *Distance from Support to "Centre"*

FIG. 2.

It then follows that:

$$B=\frac{1}{6}wh_1^2-C_1B=\left[\frac{\frac{l}{2}-\frac{h_1+h_3}{3}}{l}\right].\,wh_2^2-C_2B$$

$$=\frac{1}{6}wh_3^2-C_3B=\left[\frac{\frac{l}{2}-\frac{h_1+h_3}{3}}{l}\right].\,wh_4^2-C_4B$$

Now introducing the "virtual" height, length, and breadth defined by the following equations:

$$h_v=\frac{h_1}{\sqrt{(1+C_1)}}=\frac{h_3}{\sqrt{(1+C_3)}}\text{ and}$$

$$b_v=\frac{b}{\sqrt{(1+C_2+\sqrt{(1+C_4)}}};$$

$$l_v=\frac{l}{\sqrt{(1+C_1+\sqrt{(1+C_3)}}}$$

The above equations may be written as follows:

$$B=\frac{1}{4}wb_v^2.\frac{\frac{l_v}{2}-\frac{2}{3}h_v}{l_v}$$

$$\text{or}\quad h_v=\frac{b_v}{2}\left(\sqrt{3=\left(\frac{b_v}{l_v}\right)^2}-\frac{b_v}{l_v}\right)$$

It is interesting to compare this result with a second degree parabola with closing line representing the bending moment in an ordinary beam uniformly loaded and with partially fixed ends. Provided the bending moment at the centre is B, and at the two supports C_1 .B and C_3. B, the distance between the two points of contraflexure is exactly:

$$l_v=\frac{l}{\sqrt{(1+C_1)}+\sqrt{(1+C_3)}}.$$

The results obtained above may therefore be expressed as follows:—

The bending moment in a slab, the edges of which are partially fixed, may be found by substituting for the length and breadth of the slab a virtual length and breadth, such being the distance between the points of contraflexure of a beam spanning in one direction only, similarly supported and fixed.

It is profitable here to note what the reduction would be for various cases of fixing. The graph (*Fig.* 2) shows the "virtual span" for various degrees of fixing.

$$\frac{Bs}{Bc}=\frac{\text{Bending moment over support}}{\text{Bending moment at centre}}$$

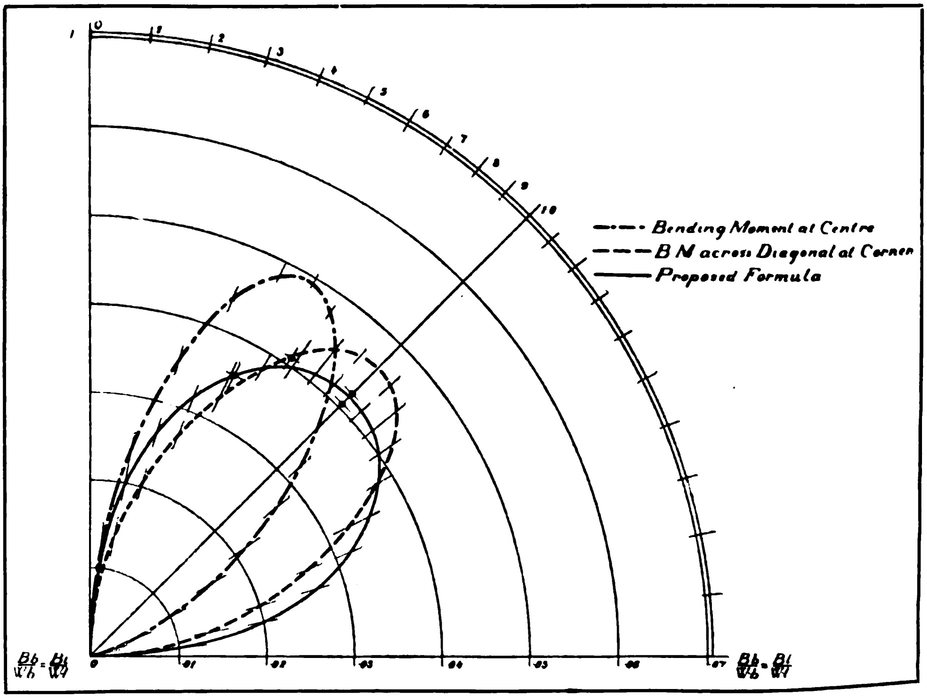


FIG. 3.

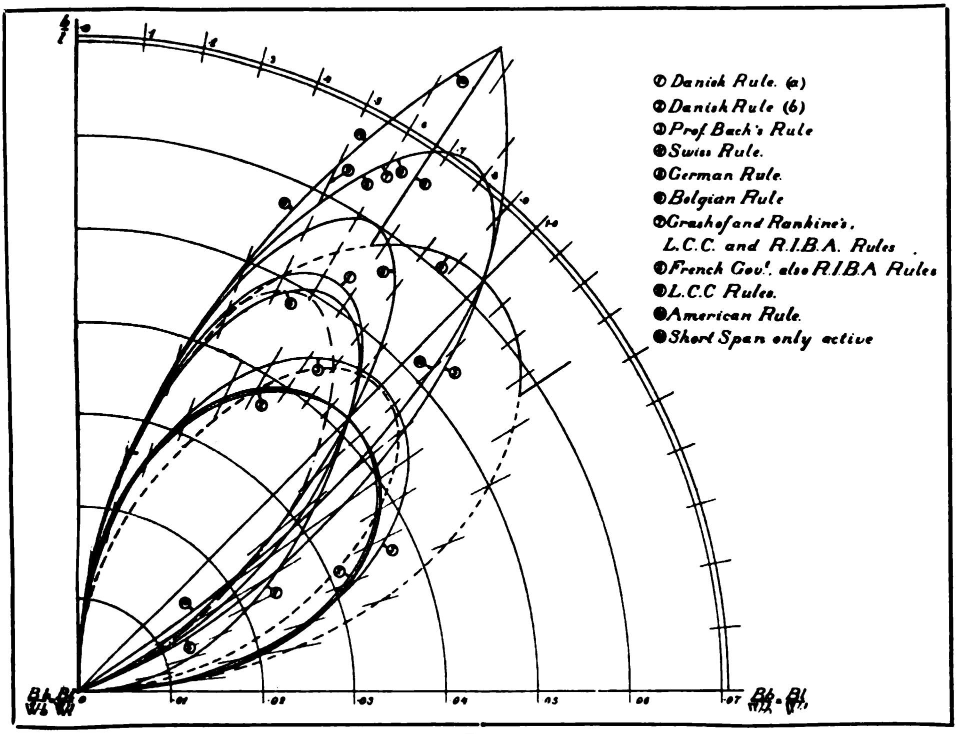


FIG. 4.

The two outside scales represent the supports, and connecting by a straight line a point on one with a point on the other, the virtual length l_v may be read directly on the centre scale. The distance from the support to the "centre" (i.e., the point where the maximum bending moment occurs) is read as sc/l_v on the inside of the outer scales.

It is worth noting that the effect of fixing one edge is to shift the "centre" of the slab towards the opposite edge. The position of this "centre" (point of maximum bending moment) for various kinds of fixing may also be obtained from the table.

It was pointed out at the beginning that a formula ought to provide such dimensions of concrete and reinforcements as would ensure the same factor of safety as that used for other structural members of reinforced concrete. The next step, therefore, is carefully to compare the results of tests on slabs with tests on beams so supported that the usual manner of calculation applies. Of these there exists a very complete series carried out by Professors Bach and Graf in the years 1911 to 1914. Altogether these tests comprise 52 slabs and 35 beams, together with a number of tests on cubes and steel bars. Most of these slabs were 2 metres square, and the load was applied at points the distance between which in both directions was 50 centimetres. The load, therefore, was not absolutely uniformly distributed, but would give bending moments that were slightly in excess of what would have obtained under a uniform load.

To show graphically a comparison between the results of these tests and the results of the formula evolved above, and also the exact theoretical solution of the problem, it has been found convenient to depart from the usual practice of expressing the bending moment in terms of the shortest span of the slab (wb^2). The bending moment here has been expressed in terms of the total load on the slab (wlb). In this way the bending moment becomes the product of the total load on the panel, and a factor representing the shape of the slab. Furthermore, polar coordinates have been adopted, any radiating line being set at the same angle as that of the diagonal of the slab under consideration, while the radius represents a factor, depending on the shape of the slab, by which the total load should be multiplied to give the bending moment B in ft.-lb. per 1 ft. of section. *Fig.* 3 shows a comparison between the rule and results of tests.

It would appear from this that, although the theory gives a bending moment that should be larger for a short distance of the slab near the centre, the results of the tests would indicate that when the slab breaks the assumption here adopted of uniformly distributed bending moment is more correct.

On the diagrams shown in *Fig.* 4 a comparison is made between the rules of different countries and with the proposed formula and the exact theoretical solution. Approximate formulas for the latter are indicated by Professor M. Westergaard in his paper referred to above.

Introducing $r = \frac{b}{l}$.

Bending Moment (short span) $= Bl/l$
Bending Moment (long span) $= Bb/b$

The formulas read as follows :—

(1) Danish Rule :

$$Bl/l = \frac{W}{8} \cdot \frac{r}{1+2r^3}$$

$$Bb/b = \frac{W}{24} \cdot r.$$

(2 Alternative Danish Rule :

$$Bl/l = Bb/b = \frac{W}{24} \cdot r\frac{3-r}{1+r}$$

(3) Bach's Rule :

$$Bl/l = Bb/b = 1{\cdot}12\frac{W}{12} \cdot \frac{r}{1+r^2}$$

(4) Swiss Rule :

$$Bl/l = Bb/b = \frac{W}{8} \cdot \frac{r}{1+r^2}$$

(5) German Rule,
(6) Belgian Rule :

$$Bl/l = \frac{W}{8} \cdot \frac{r}{1+r^2+r^4}$$

$$Bb/b = \frac{W}{8} \cdot \frac{r^3}{1+r^2+r^4}$$

(7) Grashof and Rankine,
(8) French Government Rule :
(R.I.B.A.)

$$Bl/l = \frac{W}{8} \cdot \frac{r}{1+2r^4}$$

$$Bb/b = \frac{W}{8} \cdot \frac{r^3}{2+r^4}$$

(9) L.C.C. Regulations (R.I.B.A.) :

$$Bl/l = \frac{W}{8} \cdot \frac{r}{1+r^4}$$

$$Bb/b = \frac{W}{8} \cdot \frac{r^3}{1+r^4}$$

(10) American Rule :

$$Bl/l = \frac{W}{8} \cdot (1 - \tfrac{1}{2}r)$$

$$Bb/b = \frac{W}{8} \cdot \frac{1{\cdot}5r - 1}{r^2}$$

Exact Theory (Prof. Westergaard). All edges freely supported :

$$Bl/l = \frac{W}{8} \cdot \frac{r}{1+r^3}$$

$$Bb/b = \frac{W}{48} \cdot r(1+r^2)$$

Span *b* fixed,
Span *l* simple :

$$Bl/l = \frac{W}{24} \cdot \frac{r}{1 + .4r^4}$$

$$Bb/b = \frac{W}{80} \cdot r(+ .3r^2)$$

Over support :

$$\text{Max. } Bs/l = \frac{W}{12} \cdot \frac{r}{1 + .2r^4}$$

Span *l* fixed,
Span *b* simple :

$$Bl/l = \frac{W}{8} \cdot \frac{r}{1 + \cdot 8r^2 + 6r^4}$$

$$Bb/b = {\cdot}015Wr \cdot \frac{1+r^2}{1+r^4}$$

Over support :

$$\text{Max. } Bs/b = \frac{W}{8} \cdot \frac{r}{1 + \cdot 8r^4}$$

All edges fixed :

$$Bl/l = \frac{W}{8} \cdot \frac{r}{3+4r^4}$$

$$Bb/b = {\cdot}009Wr(1+2r^2-r^4)$$

$$\text{Max. } Bs/l = \frac{W}{12} \cdot \frac{r}{1+r^4}$$

$$\text{Max. } B/s/b = \frac{W}{24} \cdot r.$$

DISCUSSION.

Mr. E. S. **Andrews**, in moving a vote of thanks to the lecturer, said the subject was of very great interest and importance. The President, in opening the meeting, had stated that it was a controversial subject, but he (the speaker) did not know to what extent there was any controversy about it. He had never yet seen any evidence brought forward to dispute the accuracy of the test results made by Professor Bach, and therefore since the test results agreed with the formulæ and theory expounded by the lecturer, he did not see that any evidence had been brought to challenge the accuracy of that method of treatment. Quite a number of very important points had been brought out in the paper. He wished particularly to draw attention to the point emphasized by the lecturer to the effect that, after the initial crack occurred somewhere near the centre of the slab, as the loading proceeded the material in the other portions of the slab gradually took up duty as the steel gave way in the centre. The acceptance of that particular example of a general principle was of the utmost importance to engineers interested in the matter of the factor of safety. Too many people imagined that just because at a certain point in a structure an instantaneous high stress could be obtained, the structure was in imminent danger of collapse. There was a sort of principle of borrowed strength by which one part of a structure could help the other in maintaining equilibrium, and that was of very great importance. The more structural engineering was studied, the more the importance of secondary stresses, reverse bending moments, and other matters of that kind was realised. There was a tendency on the part of some of the more conservative engineers to produce heavier design, whereas of course the whole aim of the increased scientific treatment of the subject was to produce not heavier design but lighter and more accurate design. He had been particularly interested with what might be called the circular cracks which appeared practically at right angles to the main cracks in the slabs, and would like to have information on that point.

Mr. W. A. **Green** seconded the vote.

The **Chairman** (Mr. E. Fiander Etchells, President) said in calling the paper controversial he had meant that the subject was one upon which a great variety of opinions had been expressed. He had also had in mind the polar charts exhibited, showing nine or ten different solutions of the problem. He thought in future differences of opinion would be less and less, because the appeal would be to the testing machine and the facts.

The **Lecturer**, replying to a question by Mr. Andrews, said the circular cracks had nothing to do with what he had been speaking of. When a load was put on a big panel, undoubtedly a certain amount of tension in all directions was set up in the bottom of it. It was like forming an inverted dome of it, and the top was trying to push away. All portions of the slab other than the corners were tied together by a sort of circular action, while the corners, not being so held, were being pushed away from the main body of the slab. Setting up a large amount of tension in the bottom of the slab was the same as a certain amount of bending moment in the slab, and having no steel in the top the corners would try to crack up.

Mr. **Andrews**: Is it not a sort of reverse moment effect?

The **Lecturer**: Yes. There is always a tendency for the corners to lift, and if they are prevented from doing that cracks result.

Mr. M. E. **Yeatman** said what had struck him was the startling simplicity of the formula, and the finding of the value of k—which was the important point apparently—from the theories laid down. It all seemed to depend on the assumption that the bending moment along the middle and down to the corners was uniform all the way. That was rather a startling assumption.

Mr. H. K. **Dyson** said the paper was based on the assumption that the bending moment was uniform along the lines of fracture; he was rather in sympathy with it as a whole. The cracking occurred first at the centre where the stress was at a maximum, but when the centre had cracked the adjoining portions began to take up the same amount of work. They developed a bigger bending moment as the cracking proceeded, and the lines of fracture at rupture were more or less triangular at the ends. He did not think the lecturer's formula told the whole story, but he thought the bending moment could not be uniform along the whole of the line of rupture. He thought the bending moment was fairly uniform along most of the lines, but not of the same intensity right up in the corners—in the diagonals. In fact it had been pointed out that the corners were bending in a reverse manner; therefore the bending moment was hardly likely to be the same in those corners as in the middle. The formula was only an approximation to the truth, though he thought it was a closer approximation than the so-called exact theory. He thought the author's suggestion of tension in the corners producing a sort of reverse flexure was quite a good one. He also thought Mr. Yeatman had made another point, which was that there was a short beam action across the corners so that the slab was carried more or less on a sort of circular framework or circular beams. It was not carried purely on the rectangular edges, but by a sort of cross beams at the corners. Another point was that where a slab was partly fixed over supports the nature of the stresses was altered very considerably. The author proposed to set off the points of contraflexure, and then calculate the bending moment between those points of contraflexure just as if it was an ordinary rectangular slab.

Mr. FORCHAMMER said the lecturer had mentioned that the Danes had two rules. As a member of the committee which had considered the matter he could say this was because they could not agree; but they could agree that both rules could be used.

Mr. A. GRAHAM said the discussion was bringing up the theory of plastic deformation again. When stresses were put on any material beyond its limit it began to fail. It began to take up stresses, and that made a great difficulty in calculating and setting down a theory, because during the duration of the load features were changing every minute in the material under the load. There were two or three explanations of the matter. Mr. Dyson believed in his plastic deformation theory, and was supported by Mr. Yeatman to a certain extent because Mr. Yeatman also showed that when the slab began to fail in the middle the load began to flow down the diagonals.

Mr. H. K. G. BAMBER said he would like to ask one or two questions from the practical point of view. He gathered that all these questions of reinforcement, and the very difficult and intricate calculations that had to be gone through to establish the proper amount of reinforcement for a slab, were based somewhat on the compression strains which could be sustained by the cement, which formed a very important part of that slab. He would like to know whether the various formulæ for reinforcement were subject to alteration or revision as the result of the known fact that the cementitious material which had been made some few years since was practically unknown to-day? That cement had had a tensional stress of 450 lb. per sq. in. in seven days, but the strains obtained by the cement of to-day were practically 100 per cent. greater than that; and he would like to know whether that fact had any bearing upon the position in which the reinforcement should be placed, and its amount.

Mr. KRUSE said he thought the theory of the paper was excellent, but the matter ought to be looked at from a practical point of view. The slab should be so constructed that it would not form any crack. It would not answer to have superfluous strength along lines which could not be utilised until the slab had actually cracked. If an architect complained of a crack along a floor it was no use to reply that the crack did not matter because if the floor were loaded the load would be taken by other points. Practical people wanted a floor that would not crack.

Mr. W. G. PERKINS said he had come across many concrete slabs that had cracked; in fact, nearly every slab that he had seen constructed had cracked at some time or other—not always at right angles to the reinforcement but parallel to the reinforcement. He did not know why the cracks occurred, but perhaps they were due to shrinkage in the concrete itself. Definite cracks appeared in the portions of the concrete between the rods; if the cracks had appeared in the other direction one would have said that something was overstressed. Before they could develop formulæ they had to make certain assumptions. They never really knew whether those assumptions were correct, and that led him to think that it was far better to assume that there was not continuity in a slab, and to put in the full amount of reinforcement in the shorter direction. A workman who had no knowledge of stresses in constructing a slab invariably placed pieces of steel across the corners, as instinct taught him that that had a valuable effect in strengthening the slab.

The Lecturer, replying, said the principal thing was, could they accept the definition of the factor of safety as being the ultimate load divided by the working load? If the factor of safety was thus defined, practiclly everything else followed. They then had to study the condition at failure, and at failure at all events there was no doubt that the stresses did actually distribute themselves equally until they came right down in the corner. That was because steel had the property of having a proportionate limit, after which elongation took place with little increase in the stress. Mr. Kruse wanted a factor of safety introduced against the first crack; if that were done there would be so much steel in the structure that it would not be useful for practical purposes. Further, there would be no uniform rule to go by, because so many things played a part in the appearance of cracks.

CONCRETE AGGREGATES—*continued.*

Norfolk—(Snettisham Beach).

(1) *General description :* Beach shingle.
(2) *Source and locality of same :* Snettisham Beach, nr. King's Lynn.
(3) *How obtained :* Loaded by hand to vessels.
(4) *From whom obtained?* Sowerfeld & Thomas, Ltd., King's Lynn.
(5) *Is available quantity limited? If so, how?* Practically unlimited.
(7) *Transport facilities :* Vessel to King's Lynn.
(8) *Is there any provision at or near source for washing or crushing?* No.
(9) *Price per cubic yard, and where delivered :* Fluctuates, but about 9*s.* 11*d.* per ton.
(10) *Is composition uniform?* Yes.
(11) *Detailed description :—*
    (*a*) *Kind of stone or coarse material :* Flints (about 1 in. to 2½ in.)
    (*b*) *Kind of sand or fine material :* Sharp sea sand and fine flints, ¼ in. to ¾ in.
    (*c*) *Relative proportions of coarse and fine material :* Approx. half and half, if through a ¾ in. screen.
    (*d*) *Shape of particles* (*rounded, angular, flat, etc.*) *:* Rounded.
    (*e*) *Size of particles ; approximate percentage that needs crushing to pass ¾ in. screen :* About 50 per cent.
    (*f*) *Impurities present* (*as clay, sulphur, coal, organic matter*) *:* None.
(12) *Is sample sent with this sheet?* Can be submitted if required.
(13) *General remarks :* Used for concrete for many years.

Norfolk—(Stoke Ferry).

(1) *General description :* Flint gravel.
(2) *Source and locality of same :* Stoke Ferry.
(3) *How obtained :* By digging and sifting.
(4) *From whom obtained?* Wereham Gravel Co.
(5) *Is available quantity limited? If so, how?* Very limited.
(6) *Present maximum output per day* (*stated in cubic yards*) *:* Six.
(7) *Transport facilities :* G.E.R., 1¼ miles.
(8) *Is there any provision at or near source for washing or crushing?* No.
(9) *Price per cubic yard, and where delivered :* 4*s.* 6*d.*
(10) *Is composition uniform?* No.

(11) *Detailed description :—*
    (*a*) *Kind of stone or coarse material :* Flint.
    (*b*) *Kind of sand or fine material :* Flint siftings.
    (*c*) *Relative proportions of coarse and fine material :* 80 per cent. coarse, 20 per cent. fine.
    (*d*) *Shape of particles* (*rounded, angular, flat, etc.*) *:* Rounded.
    (*e*) *Size of particles ; approximate percentage that needs crushing to pass ¾ in. screen :* From 6 in. to ¾ in.
(12) *Is sample sent with this sheet?* No.

Norfolk—(Strumpshaw).

(1) *General description :* Shingle, sand.
(2) *Source and locality of same :* Strumpshaw, Norfolk.
(3) *How obtained :* Gravel workings.
(4) *From whom obtained?* Mr. Edward J. Edwards, contractor, Norwich.
(5) *Is available quantity limited? If so, how?* 1,000,000 yds. or more.
(6) *Present maximum output per day* (*stated in cubic yards*) *:* No.
(7) *Transport facilities :* Road traction-engines and horses.
(8) *Is there any provision at or near source for washing or crushing?* No.
(9) *Price per cubic yard, and where delivered :* Dependent on screening and labour ; load on rail Lingwood, G.E.R.
(10) *Is composition uniform?* No.
(11) *Detailed description :—*
    (*a*) *Kind of stone or coarse material :* Flint.
    (*b*) *Kind of sand or fine material :* Sharp silica.
    (*c*) *Relative proportions of coarse and fine material :* Screened to meet requirements.
    (*d*) *Shape of particles* (*rounded, angular, flat, etc.*) *:* Rounded.
    (*e*) *Size of particles ; approximate percentage that needs crushing to pass ¾ in. screen :* As jamb varies, and contained stone up to 12 in., material screened to requirements.
    (*f*) *Impurities present* (*as clay, sulphur, coal, organic matter*) *:* Virtually nil.
(12) *Is sample sent with this sheet?* No.

Norfolk—(Thorpe Abbots).

(1) *General description :* Gravel flints.
(2) *Source and locality of same :* Thorpe Abbots Pit, Thorpe Abbots.
(3) *How obtained :* Manual labour.
(4) *From whom obtained?* Dr. F. N. Kay-Menzies, 37, Egeton Gardens, London.
(5) *Is available quantity limited? If so, how?* Not tested ; depth of face, 13 ft. ; acreage, 12.
(6) *Present maximum output per day* (*stated in cubic yards*) *:* 10 yds. per day.
(7) *Transport facilities :* Road haulage.
(8) *Is there any provision at or near source for washing or crushing?* No.
(9) *Price per cubic yard, and where delivered :* 5*s.* in pit.
(10) *Is composition uniform?* Yes.
(11) *Detailed description :—*
    (*a*) *Kind of stone or coarse material :* Grey flints.
    (*b*) *Kind of sand or fine material :* No builder's sand.
    (*c*) *Relative proportions of coarse and fine material :* 1 of coarse to 5 of fine.
    (*d*) *Shape of particles* (*rounded, angular, flat, etc.*) *:* Rounded.
    (*e*) *Size of particles ; approximate percentage that needs crushing to pass ¾ in. screen :* About 60 per cent.
    (*f*) *Impurities present* (*as clay, sulphur, coal, organic matter*) *:* About 15 per cent. loam.
(12) *Is sample sent with this sheet?* No.

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	6
Clean Shingle, ¾ in. mesh	,,	14	6
,, ,, ½ in. mesh	,,	15	6
Best British Portland Cement	per ton 63s. to	68	0
"Ferrocrete" Portland Cement	10s. per ton extra.		

Boarding for Shuttering—		Sawn.		Wrot.	
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

Sawn Timber for Struts and Supports—	
3 in. by 4 in.	from £20 per standard.
3 in. by 6 in. and 3 in. by 7 in.	,, £22 ,,

Mild Steel Rods for Reinforcement—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	6
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds.	,,	14	6
¼ in. Rounds.	,,	16	6

MATERIAL AND LABOUR INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

Portland Cement Concrete well mixed with Clean Water in proportion.

1 : 2 : 4—				s.	d.
Do.	do.	in foundation	per yard cube	45	0
Do.	do.	in columns	,, ,,	50	0
Do.	do.	in beams.	,, ,,	50	0
Do.	do.	in floor slabs 4 in. thick	per yard super	5	3
Do.	do.	in floor slabs 6 in. thick	,, ,,	7	9
Do.	do.	in floor slabs 9 in. thick	,, ,,	11	3
Do.	do.	in walls 6 in. thick	,, ,,	8	0

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

Steel Reinforcement, including Cutting, Bending and putting into Position and securing with Stout Binding Wire—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	26	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

Shuttering—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	60	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	50	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11
Do. do. as last in narrow widths	,, ,,	1	1
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1
Do. do. as last in narrow widths	,, ,,	1	3
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

The MIXING and PLACING of CONCRETE

To be carried out on money-making, time-saving lines, necessitates the employment of the best plant obtainable. No other than the best is of any real value.

The "ZENITH" CONCRETE MIXER,

In conjunction with our Chuting Plants, offers the most efficient and economical method of : : approaching all concreting problems. : :

With a run-about machine like the "Zenith-Pup" for small work, the range of the "Zenith" models provide for every class of contract.

Their Chief Points are:

SIMPLE, YET EFFECTIVE DESIGN,
ECONOMY AND RANGE OF POWER,
LARGE OUTPUT OF MIXED CONCRETE,
PERFECT MIXING OF THE AGGREGATE,
NO WASTE OR SPLASHING,
ECONOMY OF WATER,
ADAPTABILITY TO ANY FORM OF DRIVE.

Our Expert Engineers will be glad to collaborate with you in devising the best solution to your difficulties.
Our Service is at your Command.

Concrete Machinery Department:

The BRITISH STEEL PILING CO.

DOCK HOUSE,
BILLITER ST.,
LONDON, E.C.3.
Works:
Claydon, Suffolk.

MEMORANDA.

Joint Filler for Concrete Roads.

A COMPOSITION recommended by the U.S. Bureau of Public Roads for filling lines caused in concrete roads by joints and cracks has the same colour as the concrete, and the surface treated has the appearance of a continuous and unbroken slab. The compound consists of one part of crude rubber, twelve parts of resin, and sufficient barium sulphate to impart the desired tint. The rubber is first dissolved in petrol and the resin is mixed in with the application of sufficient heat, the colouring material being added as required. Any desired consistency can be obtained by varying the proportions. The compound has remarkable adhesive properties, and is applied after having been heated so that it will readily flow into joints and cracks.—*The Times*.

Reinforced Concrete Pit Linings.

IN his presidential address to the North Staffordshire Institute of Mining Engineers last month Mr. F. E. Buckley said a great deal had been done with regard to the use of reinforced concrete lining of main roadways, but it might be adopted to a much greater extent. It was the ideal method of lining, because it was more permanent, presented a smooth surface on which coal dust would not adhere to any extent, was easily brushed clean, and, if systematically whitewashed, improved the illumination, tending to greater safety generally.

Concrete-and-Rubber Roads.

FOLLOWING experiments by Mr. E. F. Spurrell, Borough Surveyor, the Holborn Borough Council has decided to lay an experimental section of carriage-way in St. Martin's Lane with concrete blocks covered with rubber. The concrete blocks will be of the same size as wood blocks, and on these a pad of rubber ¼ in. thick will be affixed. Portland cement will be used as the binding medium for the blocks. It is anticipated that this type of paving will last twice as long as wood blocks.

A Technical Assistants' Employment Bureau.

THE formation by the Architects' and Surveyors' Assistants' Professional Union of a Central Employment Bureau available for use by all technical assistants in the building industry (assistants in the offices of architects, surveyors, reinforced concrete specialists, builders, etc.) has filled a long-felt want. The Bureau has been recognised by several of the professional institutions, and the Royal Institute of British Architects and the Society of Architects have both decided to pass to the new Central Bureau all vacancies of which they are notified.

Fire-Resistance of Concrete Post Caps.

THE Chicago Bureau of Underwriters has carried out a series of tests with a view to ascertaining the best means of protecting timber columns from fire, from which it has come to the conclusion that the fire-resistance of 12 in. by 12 in. columns of usual length of Douglas fir or southern pine is practically doubled if the ends are adequately protected. Several methods of protecting the ends of the columns were tried, and the conclusion arrived at that the best is a reinforced concrete cap. The caps used in the experiments were made of 3 parts cement, 2 parts sand, and 4 parts crushed limestone to pass a ½ in. mesh. The reinforcement consisted of ¼-in. rods, bent by hand, and welded at all surfaces of contact. The weight of the caps for a timber column 10 ft. 10 in. long with a cross section of 12 in. by 12 in. varied from 316 to 335 lb.

Concrete Houses at Aberdeen.

THE Housing Committee of the Aberdeen Corporation has recommended the erection of a number of concrete houses at Torry the cost of which works out at £175 per house less than similar houses built of brick.

WRITE FOR THE CATALOGUE

LIMITATION of space prevents us from describing in detail the full particulars of the Victoria Concrete Mixer and our many other mixing and blockmaking machines. We have endeavoured, from time to time, to explain the principal features, one at a time, in these advertisements: but we are always glad to send you a catalogue, which gives the fullest information concerning the machine in which you are most interested.

Every user of concrete or tar macadam should send for a copy of our catalogue covering the mixers; we are only waiting for your name and address.

When writing, please say in which machine you are most interested, the large Victoria Concrete Mixer, the petrol-driven Victoria Mixer, the hand-driven Victoria Mixer, the Smith Mixer or the Combined Drying and Mixing Plant for tar macadam—it will reach you by return.

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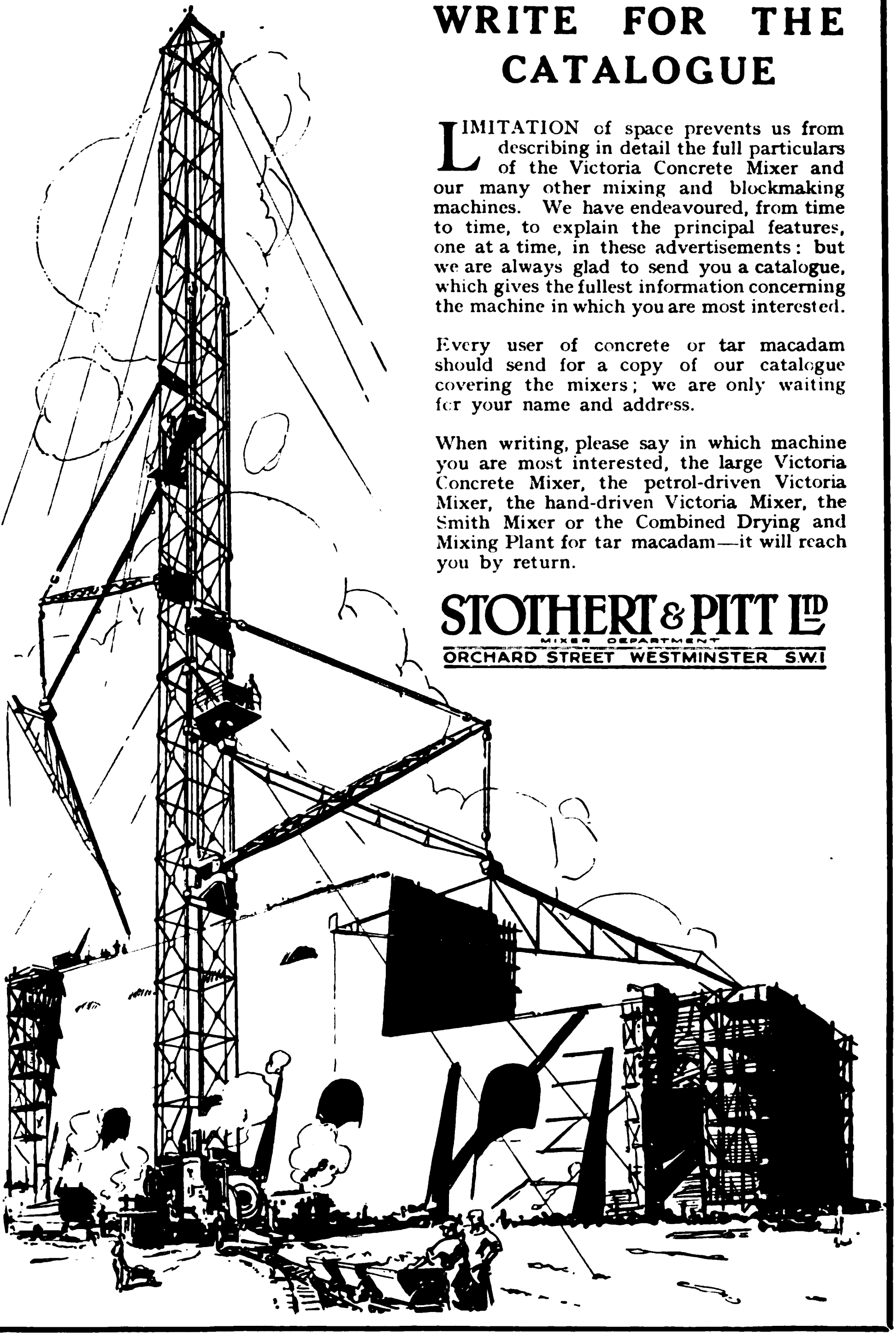
MIXER DEPARTMENT
ORCHARD STREET WESTMINSTER S.W.I



PROSPECTIVE NEW CONCRETE WORK.

Barnsley.—*Road.*—The Corporation has received the sanction of the Ministry of Health to borrow £55,000 for the construction of a new road between Wombwell and Carlton.

Berwick.—*Pier, etc.*—It is stated the approval of the Unemployed Grants Committee has been obtained for the erection of a south pier at Berwick Harbour, and a retaining wall along the river, at an estimated cost of £47,300.

Birmingham.—*Road.*—The local authorities interested in the construction of a new main road linking Birmingham with Wolverhampton have decided to press forward the scheme. The estimated cost is £450,000, of which the Government is prepared to contribute half.

Birmingham.—*Sewage Works.*—The Birmingham Tame and Rea District Drainage Board has agreed to borrow £34,000 for the enlargement of the Storm-water tanks at the Nechells Works.

Bolton.—*Road.*—The T.C. has adopted a scheme for the construction of a road connecting Chorley New Road with Wigan Road, estimated to cost £152,437.

Burntisland.—*Dry Dock.*—The Burntisland T.C. is considering a proposal to construct a dry dock at Burntisland at an estimated cost of £82,000.

Burslem.—*Sewage Works.*—The Ministry of Health has sanctioned loans amounting to £19,320 for extensions at the Burslem sewage works.

Canvey.—*Bridges.*—The Rochford R.D.C. has adopted plans for the construction of a bridge over the Creek, and for the construction of a new roadway from the main road to cross the railway by means of a bridge, and to link with the proposed new approach to the Creek bridge. The total estimated cost is £47,000.

Carrick-on-Suir.—*Houses.*—It is stated that owing to the high price asked for the construction of the workmen's houses in brickwork, the Urban Council proposes to build them of concrete blocks.

Deal.—*Concrete Pavilion.*—The T.C. is considering the erection of a concrete pavilion on the sea front.

Doncaster.—*Sewage Works.*—The Ministry of Health has given its sanction to the Corporation to borrow sums amounting to £52,355 for sewerage and sewage disposal works.

Dorking.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the U.D.C. for sanction to borrow £2,980 for sewerage and sewage disposal work.

Dysart.—*Harbour Works.*—The T.C. is considering a proposal to spend £50,000 on improving Dysart Harbour.

Ferry Fryston.—*Waterworks.*—The Ministry of Health has held an inquiry into an application by the Pontefract R.D.C. for sanction to borrow £18,000 for the provision of waterworks. It is intended to construct a reservoir to hold over 100,000 gallons.

Golcar.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application for sanction to borrow £3,270 for the extension of the Appleyard sewage works.

Greenock.—*Swimming Pond.*—The Scottish Board of Health has given its sanction to the Corporation to construct a concrete open-air swimming pond on the shore at Battery Park.

Haverfordwest.—*Bridges.*—The R.D.C. has decided to co-operate with the Pembroke R.D.C. in reconstructing Guildford Bridge. The Council has also decided to reconstruct Broadhaven Bridge.

Hexthorpe.—*Open-air Bath.*—The Doncaster Corporation proposes constructing an open-air swimming bath at Hexthorpe.

Holmfirth.—*Sewage Works.*—The U.D.C. has received sanction to borrow £13,340 for alterations to the sewage disposal works.

Lancing.—*Road Works.*—The West Sussex C.C. has approved a scheme for the reconstruction of the main road between Norfolk Bridge and Lancing at an estimated cost of £11,400.

Llandudno.—*Open-air Bath.*—The U.D.C. is considering the question of providing open-air sea-water baths on the foreshore.

London (Bermondsey).—*Open-air Bath.*—The L.C.C. has decided to construct an open-air swimming bath in Southwark Park. The bath will be 180 ft. long and 60 ft. wide, and constructed of reinforced concrete.

Macclesfield. — *Reservoir.* — Parliamentary powers are being sought by the Corporation to construct a new reservoir costing £80,000.

Macclesfield.—*Sewage Works.*—The Corporation has decided to spend £17,000 in improvements at the Sewage Works.

Malvern.—*Tunnel.*—A proposal of the Great Western Railway Co. to construct a new tunnel under the hills to expedite traffic was brought forward at a meeting of the Malvern Hills Conservators.

Maryport.—*Harbour Works.*—The Maryport Harbour bondholders are considering a proposal to borrow £150,000 for the extension of the dock and harbour works, and the improvement of the approaches.

Overstrand.—*Defence Works.*—The Erpingham R.D.C. has resolved to apply to the Ministry of Health for sanction to a loan for sea protection works at Overstrand.

Poole.—*Reservoir.*—The Corporation has decided to apply to the Ministry of Health for a Provisional Order empowering it to construct a new reservoir at Corfe Mullen.

SEDGELEY.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the U.D.C. for sanction to borrow £16,000 for sewerage and sewage disposal works.

SKEGNESS.—*Defence Works.*—The Commissioners of Sewers have been notified that the Government is prepared to make a grant of £10,000 towards the cost of the new sea defence works at Ingoldmells, near Skegness.

SKEGNESS.—*Sewage Works.*—The Ministry of Health has approved the application of the U.D.C. to enlarge and modernise the town's sewage disposal works at a cost of £16,500.

STOCKPORT.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the U.D.C. for sanction to borrow £180,000 for sewerage and sewage disposal works at Cheadle Heath.

TILBURY.—*Dock.*—It is stated that the Port of London Authority has acquired land for the building of another dock on the north side of the Royal Albert Dock, and has decided to spend £14,000,000 on modernising the port and works.

WEST HARTLEPOOL.—*Road.*—At a meeting of the T.C. the necessity for proceeding with the relaying Burn Road with reinforced concrete was brought forward. The estimated cost is £5,500.

WIGSTON.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the U.D.C. for sanction to borrow £18,000 for sewerage and sewage disposal works.

WOODFORD HALSE.—*Sewage Works.*—The Daventry R.D.C. has received sanction from the Ministry of Health to borrow £3,600 for the extension of the sewage works.

YORK.—*Concrete Wall.*—The City Council has decided, subject to the approval of the Unemployed Grants Committee, to proceed with the reconstruction of Foss Bank and erection of a concrete wall, at a cost of £6,800.

TENDERS ACCEPTED.

BRIDLINGTON.—*Reinforced Concrete Raft.*—The R.D.C. has accepted the tender of Mr. J. Sawden, High Street, Bridlington, at £185, for the construction of a reinforced concrete raft on the bridge over Earls' Dyke, near Fraisthorpe.

BROMSGROVE.—*Bridge.*—The Worcestershire C.C. has accepted the tender of Messrs. J. and A. Brazier, Bromsgrove, at £1,002, for the rebuilding of Dyers Bridge in reinforced concrete.

PONDERS END.—*Sewers.*—The Enfield U.D.C. has accepted the tender of Messrs. W. and C. French, Buckhurst Hill, at £7,329 10s. 3*d*., for the construction of 24 in. and 30 in. concrete pipe sewers, together with manholes and other works.

RICHMOND.—*Foundations.*—The T.C. has accepted the tender of Messrs. Speechley & Smith, at £233, for the concrete foundations at the public baths.

RECENT PATENT APPLICATIONS.

187,286.—H. Arquint and Melcator Akt. Ges.: Reinforcement for ceilings, walls and roofs.

187,289.—F. Meldrum: Construction of buildings.

187,362.—G. Schloesser: Manufacture of mortar from town refuse.

187,676.—I. Willgoose: Hollow concrete walls.

187,751.—J. M. Hall: Sectional ferro-concrete dwellings.

187,754.—M. W. Harvey: Moulding machines or presses.

187,834.—E. Rochester and A. Ingham: Walls and concrete blocks.

187,862.—F. T. Gray and R. G. Whitaker, Ltd.: Tamping mechanism for concrete block-making machines.

188,061.—A. Hiley: Pressure-fluid operated hammers for pile extraction.

188,185.—K. Wessel and A. H. Heimbach: Building slabs and buildings.

188,249.—A. A. J. D'Have: Reinforced concrete.

188,406.—J. Woolcock: Floor construction.

TRADE NOTICES.

An amalgamation has been formed between the asbestos-cement branch of Messrs. Bell's United Asbestos Co., Ltd., and the British Everite and Asbestilite Works, Ltd. The new concern will trade under the title of Bell's Poilite and Everite Co., Ltd., with offices at Southwark Street, S.E.1, and Peter Street, Manchester, and it is claimed that it is the largest producing source of asbestos-cement in the world.

Glenboig Road, Airdrie, a single-course reinforced concrete road 6-in. thick, was constructed in August, 1921, and traffic since then has included the passage of a 16-ton tractor engine with steel tyres several times per day. Underlying the road there is a bed of peat bog 40 ft. thick, but there is no sign of subsidence, cracks, or wear in the concrete. The road was constructed under the direction of Mr. W. A. Chapman, County Road Surveyor, and reinforced with B.R.C. Fabric.

Single Copies, 1s. 6d. nett. *Annual, Post Free, 18s.*

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

FEBRUARY 1923 VOL. XVIII. No. 2

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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By J. SINGLETON-GREEN, B.Sc., A.M.Inst.Struct.E., M.Am.C.I.

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Concrete Cottage Building.

Large New Manchester Warehouse.

Cement Notes. **Large Concrete Bridges.**

Questions and Answers Relating to Concrete.

The Architect and Engineer.

Data for Pricing Reinforced Concrete Work.

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The late King Edward VII.

By Appointment to
H. M. King George V.

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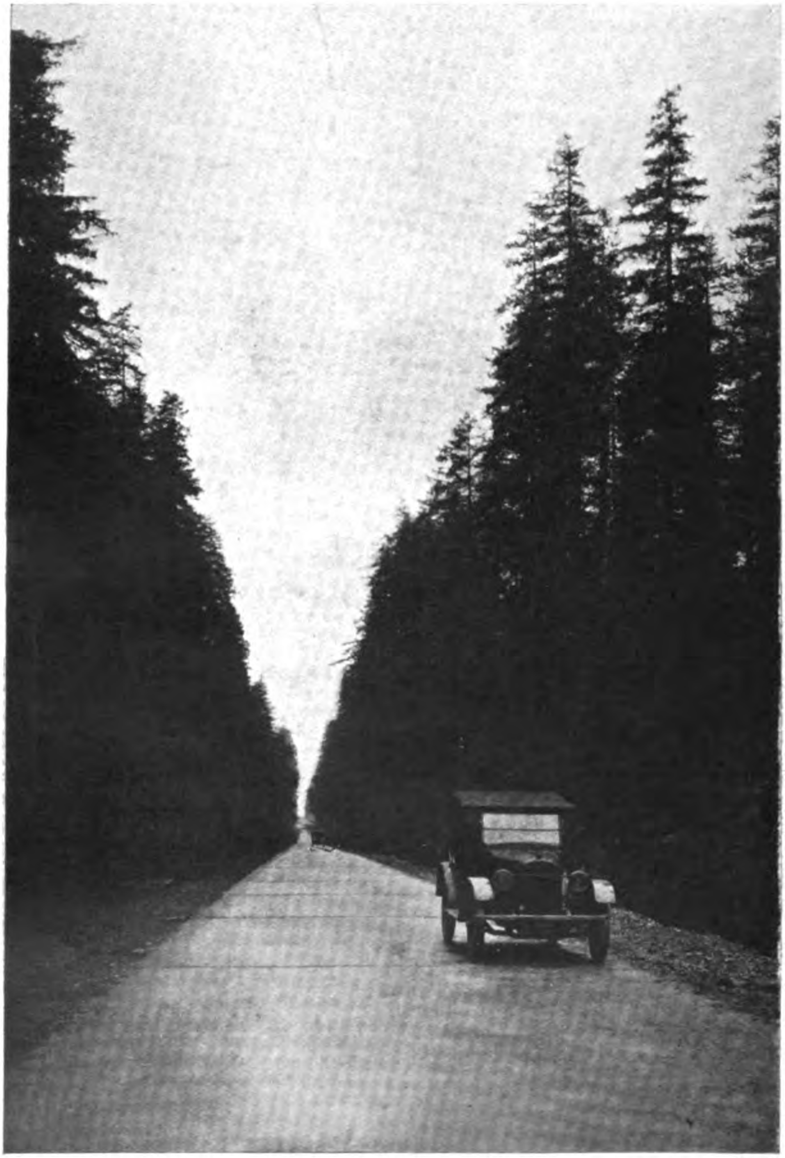
Initial cost lower than any other, and BRITISH throughout.



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—light and *extra* portable, self-contained, strongly built for long service, a high-grade RANSOME machine at a surprisingly low price.

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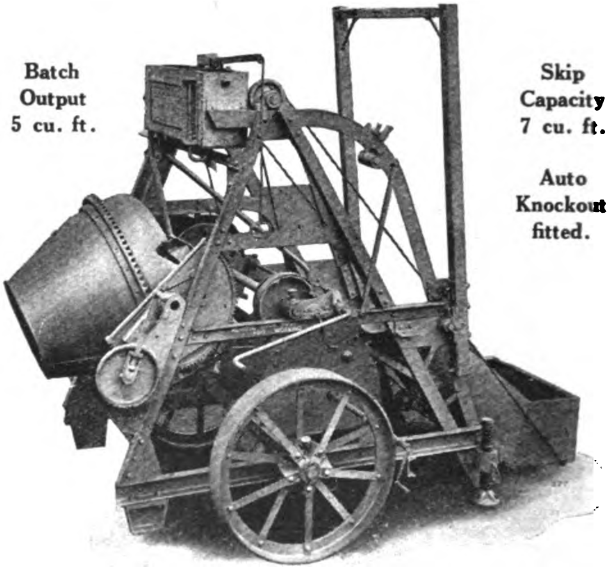
Ask Dept. B for the New Catalogue No. 28

which gives full details and photographs of the 1923 Ransome Lightweight Mixer.

If interested in larger sized Mixers ask for Catalogue No. 151, and Hoisting and Placing Equipment Catalogue No. 16.

Ransome Lightweight Concrete Mixer.



Batch Output 5 cu. ft.

Skip Capacity 7 cu. ft.

Auto Knockout fitted.

Output: 40 batches of 7·5 cu. yds. per hour.

THIS is not a cheap machine that will last only a month or two, and it is not so low-priced as some of that kind. But quality, life, efficiency and output considered, this new 1923 Ransome Lightweight is easily the best value on the market at anything near its price. . . . Fitted with a high-grade, reliable, British made 3 h.p. Petrol Engine, which is included in the Mixer Guarantee of Efficiency.

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EDITORIAL NOTES.

NATIONAL ECONOMY.

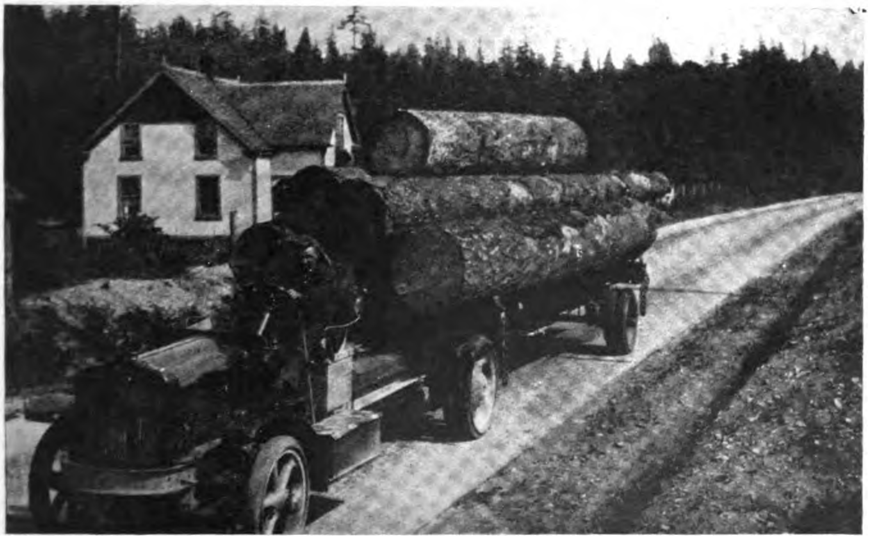
It is indisputable that the quality of the Portland cement on the market to-day is greatly superior to that of twenty or thirty years ago, and it would hardly be exaggeration to state that the modern product has double the strength of its forerunner. It is equally certain that, although there is still much to be learnt by the average user in the employment of concrete aggregates, the cleanliness and grading of the latter now receive more attention than they did in the past. Also, the present generation of contractors' foremen and clerks of works have a better knowledge of concrete and its proper method of production than did their predecessors. As the result of these advances it is right to expect that modern concrete should be a considerably better material than that laid down in 1900 or before, and this expectation is undoubtedly realised. But it is also pertinent to inquire whether full advantage is being taken of the improved concrete of to-day, because if not an opportunity for the national economy that is now of such vital importance is being lost. When we recall certain modern examples of the application of concrete, such as concrete bridges, concrete domes, and concrete skyscrapers, we readily admit that engineers have made full use of the wonderful properties of concrete, but, on the other hand, we doubt whether the majority of concrete work is being done with that degree of economy which is rendered possible by the improved quality of the modern product. It is safe to say that a modern concrete is twice as strong as one made twenty years ago with the same proportions and at the same age, but is the possible economy of this improvement being used by reducing the proportion of cement in the concrete or the dimensions of the concrete members ? Our national conservatism will render any change in this direction a slow one, but we are faced with the fact that such economy does not appear to be in the interests of those immediately concerned. The engineer may hesitate to assume responsibility in this direction. The cement manufacturer cannot be blamed if he does not recommend his customers to use weaker mixtures; it is really beyond his province, and he, quite properly perhaps, contents himself with giving information regarding the strength of his manufactures. It is only the contractor who is not bound by specifications who will seize on the possibilities of economy now available. Cheaper concrete means, however, in the end an extended use of concrete, and if this long view is taken by those to whom it may appear altruistic for the moment they will in due course reap their reward.

HARDENING CONCRETE WITH STEAM.

RECENT experiments on hardening concrete by exposing it to the action of steam under a pressure of 128 lb. per sq. in. has attracted sufficient attention to make it desirable to point out that this method of curing concrete is not a new one. Michaelis in 1880 described the use of steam under pressure for hardening concrete and for many years lime-sand bricks have been hardened in this manner. The application of steam is only suitable for blocks of moderate dimensions and such as can be placed in a hardening chamber. The articles are made in the ordinary manner, but instead of being allowed to harden naturally they are exposed to steam under pressure for several hours and are then ready for use. Steaming saves much time, but is very costly. It ensures a more uniform hardening and reduces the risk of cracking. It renders the manufacturer independent of the weather, enables less cement to be used, and avoids the cost of keeping the concrete wet for several days. These are all excellent advantages, but unfortunately, they are more than counterbalanced, in the opinion of most manufacturers of concrete blocks, by the greater cost of treatment and the heavy capital expenditure necessary when the steam is used.

THE ROAD PROBLEM.

AN aspect of the road problem to which we have frequently drawn attention, namely, the unreasonable wear and tear to which roads are subjected by heavy solid-tyre vehicles, is once more being ventilated in the daily Press, and weight is added to the arguments in favour of restricting the use of such heavy traction by the results of investigations on the subject in Canada, which have just come to hand. In these investigations, which were outlined by Mr. C. W. Cornell before a meeting of the Canadian Good Roads Association in October last, the average crushing strength on a concrete road slab was assumed to be 2,500 lb. per sq. in. With a solid-tyre lorry travelling at sixteen miles an hour the impact on striking a 1-in. obstruction was found to be as high as seven times the static load, and from this it was taken that if the maximum static load the road would safely bear was 2,500 lb. then to allow for impact one-seventh of the weight, or 357 lb. per sq. in., was the maximum allowable static load per sq. in. Working on these lines it was calculated that the loading should not exceed 800 lb. per inch width of tyres, which would allow a load of over eleven tons for a vehicle with 14-in. tyres (the maximum size used in Canada). This basis has already been adopted by the United States National Automobile Chamber of Commerce for application to lorries with 14-in. tyres, but this does not, we think, go far enough. The speed of a vehicle has quite as much, if not more, effect on a road surface than the weight of the load. Lorries with 14-in. tyres are the exception rather than the rule, and moreover the speed at which a vehicle travels increases as its weight and the width of its tyres decreases. The more urgent problem at the present time is the smaller lorry with narrow tyres travelling at a speed of twenty miles an hour and over; the four-ton or five-ton lorry with say 5-in. tyres, which imposes much more than 357 lb. per sq. in. static load on the road and the impact of which is proportionately higher owing to the greater speed. It would seem that the point has been reached where the speed and carrying capacity of modern transport have advanced beyond the capacity of engineers to build roads to carry it without incurring an expenditure so great as to make roads to withstand present-day traffic prohibitive on the ground of cost, for it has to be borne in mind that these tests were carried out on specially designed all-concrete roads, which are undoubtedly the most suitable for carrying heavy and fast traffic. If that be so, in the interests of the public and road-users themselves something must be done to prevent road surfaces from being destroyed in the wholesale manner which is going on at the present time. If roads cannot be built at a cost within the capacity of the authorities the problem must be attacked from another angle, which must necessarily be the vehicles themselves. From the experiments referred to above and from the results of other investigations the consensus of opinion of those who have studied the matter seems to be that (leaving out of account steel-tyred traction engines, which should be prohibited altogether) the weights and speeds of solid-tyred vehicles should be restricted as follows :—Up to 6 tons, 20 miles per hour ; between 6 tons and 10 tons, 15 miles per hour ; over 10 tons, 10 miles per hour ; with a limit of a total weight of 11½ tons. These restrictions could be imposed without any hardship on the owners of vehicles, for the better roads which would result would probably more than repay for any loss of time due to reduced speed in lower upkeep costs. Some experiments of the U.S. Bureau of Standards (referred to in our issue for December last) show that the wear and tear on concrete roads caused by vehicles with pneumatic tyres is practically negligible ; restrictions on solid-tyre vehicles, together with the encouragement of research to provide large pneumatic tyres at



THE PACIFIC HIGHWAY, BRITISH COLUMBIA.

[This illustration, and our Frontispiece, show the reinforced concrete road known as the Pacific Highway between New Westminster and Blaine, British Columbia. When completed this year the road will be seventeen miles in length, of reinforced concrete throughout with no top dressing. The concrete is only 7½ ins. thick in the centre and 6½ ins. at the shoulders, but its strength and wearing properties are well illustrated by the above photograph, which gives an idea of the type of traffic it carries.]

a lower cost than at present, would therefore seem to be the best lines to work on if new roads are not to need resurfacing almost as soon as they are built.

COLOURED CONCRETE COTTAGES.

THE numerous letters we have received indicate that considerable interest has been aroused by the illustrations in colour, given in our last issue, of the concrete cottages on the Liverpool housing scheme, in which a pleasant variety has been obtained by the use of concrete blocks with coloured surfaces. One of the Associations interested in the provision of better and more attractive homes for the working classes writes: " There has undoubtedly been a considerable amount of prejudice in the past against concrete building by reason of its deadness of colour, and the coloured supplement should bring home to many people that that disadvantage has now been overcome." Excellent as is the pioneer work at Liverpool, so far the æsthetic aspect of concrete has hardly been touched. Not only in colour, but also in the ease and cheapness with which it can be moulded, concrete offers enormous possibilities to those responsible for the erection of working-class dwellings, whose aim it should be to make them as attractive as possible, both inside and out. It is to be hoped that when such houses are built again, as no doubt they will be very shortly, economy will not be the sole claim put forward for concrete construction, for there is no reason whatever why a little extra care should not be taken in the design to make them as beautiful as houses built with any other material.

NEW COLUMN AND BEAM TESTING MACHINE.

By **H. E. LANCE MARTIN, B.Sc. (Lond.), A.M.Inst.C.E., of Armstrong College (Univ. of Durham), Newcastle-on-Tyne.**

THE importance of having an accurate and at the same time a readily adaptable testing machine for large concrete columns and broad-flange T beams, or floor slabs, cannot be over-estimated. This article deals with such a piece of apparatus specially designed by the writer for the new Research Concrete Laboratory now being equipped by Armstrong College (University of Durham), Newcastle-upon-Tyne, under the Professor of Engineering, Engineer-Commander C. J. Hawkes.

One of the first essentials in making concrete experiments and tests is to be quite certain of the uniformity of the mixtures. Without this no true knowledge can be gained from the breaking of specimens, however accurately the load may have been measured. The laws governing the proportions of the materials to give a desired strength and compactness are now becoming clearer, but there is yet plenty of scope for research in this direction. This important part in the study of concrete has been amply provided for by the installation of the necessary apparatus, including a battery of sieves, balances, mechanical shaker, Vicat needle and a new cement tensile testing machine (shot type), by Messrs. W. & T. Avery, Ltd., with a maximum load of 1,200 lb.

The large concrete specimens are at present made in a shed set apart for the purpose, in which bins have been constructed to store the materials. The tops of these bins are being used as benches on which physical tests such as mechanical analysis can conveniently be made. The specimens are left to mature at the temperature of the mixing shed, which is not artificially heated. When required for testing they are removed into the Cochrane Engineering Laboratory, at one end of which the new column and beam testing machine has been erected.

Figs. 1 and 2 give two views of the machine showing one or two of the accessories, either actually on the machine or close by it, together with the steel

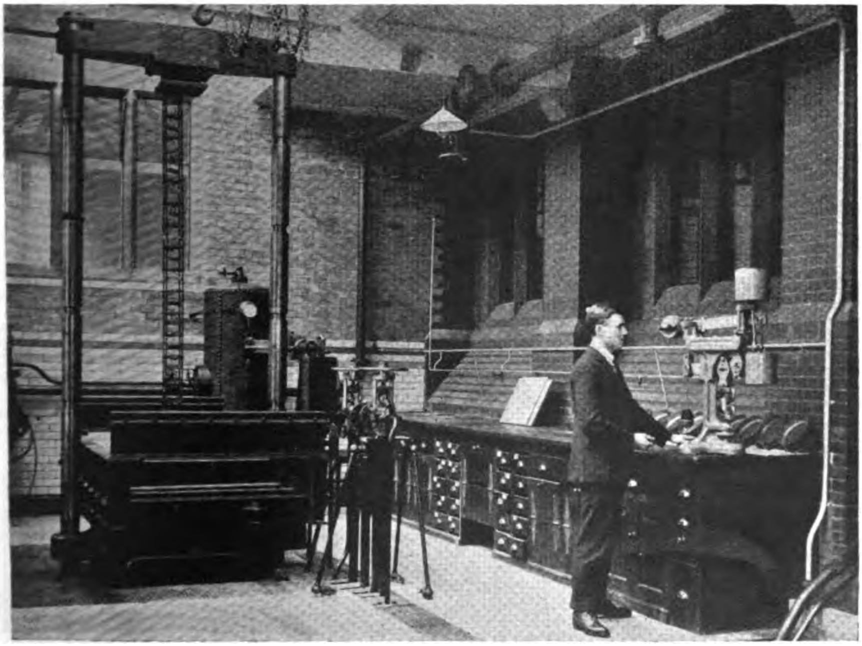


FIG. 1.

work of a reinforced-concrete column. The machine may be divided into two main parts:—

(1) A large differential hydraulic-ram for applying the load to the specimen by means of two large tension rods. The tension rods are connected together by two crossheads, the lower crosshead being fastened to the ram and the upper being capable of adjustment to convenient heights on the tension rods.

(2) A large platform connected by a system of multiple levers to a steelyard graduated to read to the maximum load of 120,000 lb., in steps of 250 lb., subdivided by means of a vernier to 25 lb. Although a much smaller load will disturb the balance of the steelyard it was deemed that this latter figure was sufficiently small to meet all possible demands for accuracy.

The length of, and distance between, the tension rods permit columns up to 10 ft. in length and beams having a maximum width of 4 ft. being placed in the machine. The spans of the beams may be varied from 1 ft. up to a maximum of 9 ft. Thus a floor slab 9 ft. by

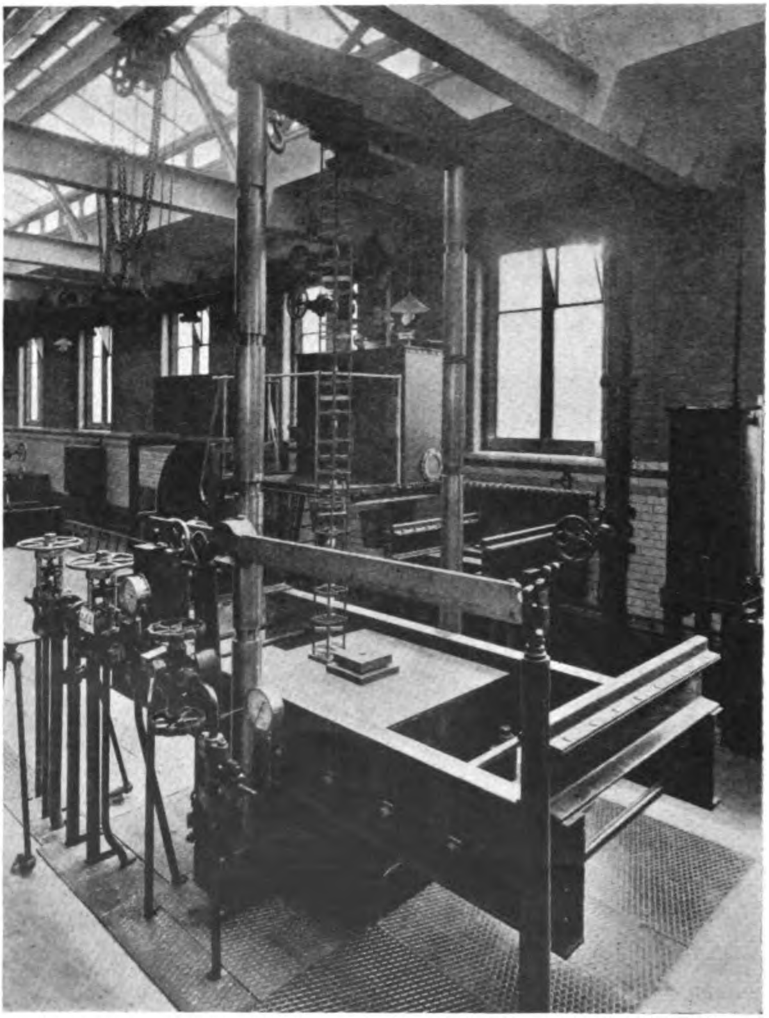


FIG. 2.

4 ft., or 36 sq. ft., can be accommodated.

In order to facilitate the placing of such heavy specimens on to the machine a Herbert Morris travelling pulley block of 2 tons lifting capacity is provided. This is seen in the photographs.

Two detachable spherical joints connect the ends of the columns to be tested to the top crosshead and the bottom platform respectively. These not only compensate for the ends of the columns not being truly at right angles to their length but allow them to bend in any direction when under load. Beams under test can be point loaded at one-third, one-quarter, or one-half span as desired.

The necessary hydraulic pressure to work the differential-ram is obtained by passing the town water supply through a "break tank" in which the pressure is reduced from that in the mains to a constant pre-determined pressure of about 25 lb. per sq. in. The water from this pressure tank is led to the underside of an intensifier, which increases the pressure approximately seventy-four times. This high-pressure water is passed through suitable piping and controlled by needle valves to the differential-ram. One of the interesting features of this hydraulic arrangement is that the operator can never overload the machine, as the pressure in the "break tank" has been so fixed that the maximum load is just reached and no more.

Fig. 3 shows the valve arrangement in a diagrammatic form. Valves 2 and 3 are linked together so that if 2 is opened 3 opens as well. Valves 4 and 6 are also linked together and act in a similar manner. The operation of opening valves 2 and 3 connects the intensifier with the hydraulic cylinder; and that of opening valves 4 and 6 exhausts the intensifier. Valves 1 and 5 are for regulating the ram, either for adjustment purposes or for putting on the load.

To the high-pressure pipes are fitted two pressure gauges. Each gauge has an additional pointer which gives the maximum pressure reached. One gauge records the pressure in the hydraulic cylinder and the other gives the pressure from the intensifier. It will be seen that if during a test the specimen gave suddenly, or the steelyard was not "floating," the reading on the gauge in communication with the hydraulic cy-

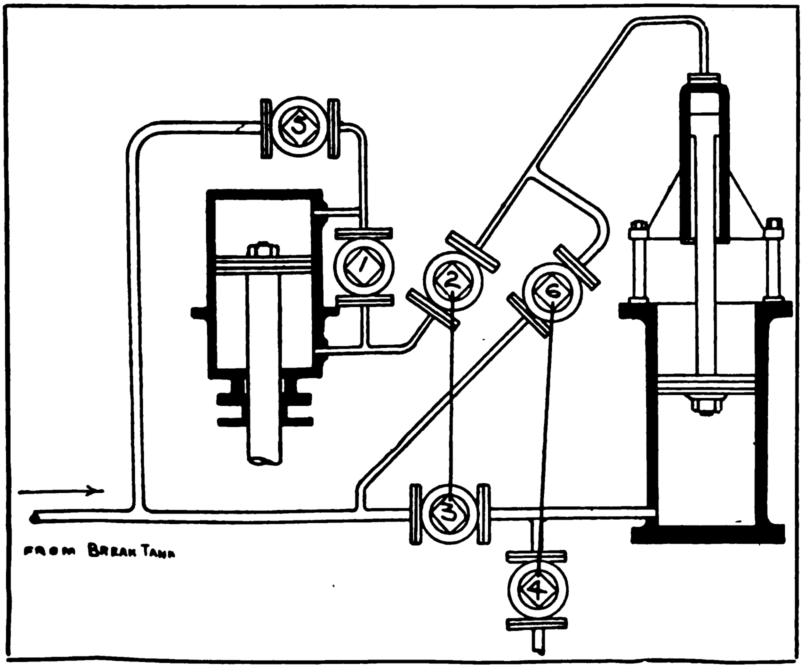


Fig. 3.

linder, as recorded by the maximum pointer, would give the required load. Moreover, if the jockey weight on the steelyard be placed at a particular load—which will, of course, cause the steelyard to rest on the bottom stop—and the pressure be applied to the ram, the rate of loading can be ascertained by watching the reading rise; consequently the loading is completely under the control of the operator.

The final tests, made after the machine was completed, demonstrated that it worked well. This is greatly to the credit of the makers, Messrs. W. & T. Avery, Ltd., of Birmingham, who took exceptional pains to make this new type of machine a success.

It is to be noted that this machine can be used for testing constructional steel columns and beams as well as those of concrete. Although the machine was primarily installed for instructional and research work it is hoped that it may be possible to undertake commercial tests.

It remains to be stated that it is due to the generosity of Dr. Cecil A. Cochrane, of Newcastle-upon-Tyne, that this work has been made possible.

BUILDING CONSTRUCTION AND EARTHQUAKES.

By GEORGE MALLET (Valparaiso).

The recent disastrous earthquakes and tidal-waves which have devastated the northern provinces of Chile have once more forcibly demonstrated the pros and cons of the different systems of building construction, and unmistakably proved the necessity of seeking modern methods of protection against the hidden dangers which menace volcanic countries or zones which suffer from periodical seismic convulsions.

The series of disastrous *terremotos* in San Francisco, Valparaiso, Kingston (Jamaica), and Messina set the technical world thinking, with the result that reinforced concrete in its present stage of development is the accepted solution of the difficult problem of escaping destruction during these terrible convulsions of Nature. Proofs of this are steadily forthcoming—one solitary edifice intact at Avezzano was of reinforced concrete; the solitary survivor recently at Copiapo was a reinforced concrete building.

Previous to the great earthquake at Valparaiso on August 16, 1906, the city had entered upon a new era of prosperity which resulted in a phenomenal boom in building construction. Handsome modern buildings were rapidly on the increase, occupying whole areas and new districts. Owing to the inevitable rise in ground values, commercial edifices begin to soar upwards in search of additional accommodation. Previous seismic disasters had been relegated to the annals of ancient history, hence the wholesale destruction and loss of life in the modern part of the city in 1906. Up to that date reinforced concrete was practically unknown in Chile. The earthquake, however, forcibly brought it to the front. The opposing prejudices of the public were then very strong, and were fostered by reports from San Francisco of people being killed by the falling outwards of the huge slabs of concrete-filling from the steel-framed skyscrapers. These slabs were held in only by the very narrow flanges on the rolled steel sections, without extra wire reinforcement. Pioneer work was of a necessity all more or less of the ferro-concrete or steel frame order, owing to the difficulty of obtaining trustworthy labour. With the introduction of steel wire mesh reinforcement the difficulties were much reduced, and the more adventurous spirits were led into the field of reinforced concrete. Progress has been so rapid that Valparaiso and Santiago can now boast of reinforced concrete buildings which from an artistic architectural point of view can hold their own with anything in the world, while from a technical point of view it is safe to assert that no edifice yet constructed can boast of a superior scientific perfection than that attained in the new building of the Anglo-South-American Bank now nearing completion in Calle Prat, Valparaiso.

REINFORCED CONCRETE AND FIRE-RESISTANCE.

V.—STRENGTH OF CONCRETE AFTER HEATING.

By J. SINGLETON-GREEN, B.Sc. (Hons. Eng.), A.M.I.Struct.E., M.Am.C.I.

VARIOUS experiments have been carried out from time to time to find the loss of strength in concrete due to heat treatment. Unfortunately, it is very difficult to get definite comparative values from these scattered reports, and the best that can be done is to compare all the results as far as possible so as to get the general ideas clearer. For instance, it would be impossible to say at this stage that the strength of basalt after a 4 hours' fire test at 900 deg. is 1·3 times that of granite after similar treatment; but we can say, however, that basalt is stronger than granite after fire. It would be unwise, if not impossible, to go beyond this point; hence it is evident that a good deal of research still remains to be done on the effect of temperature on the crushing strengths of various concretes. The following outlines indicate briefly what has been done on the subject.

BAUSCHINGER'S TESTS.[1]

In 1884–86 Professor J. Bauschinger (of Munich) made two series of fire and water tests on building columns that were loaded in a horizontal testing machine and heated by wood fire in a wrought-iron trough placed under them. Water was applied to the top surface. A test with Portland cement mortar was made on a column about 12 in. (30 cm.) square and 10 ft. (3 metres) long. The proportion of the mixture was 1 : 5 Portland cement and coarse sand. The column was tested at the age of 6½ months under a working load of 95 lb. per sq. in. It was heated for 1¾ hr. until a temperature of 600 deg. C. was attained at the middle of the sides, when water was applied. No apparent injury resulted, and when cold it was loaded to failure at 920 lb. per sq. in.

TESTS BY McFARLAND AND JOHNSON.[2]

Tests to determine the effects of fire and water on the strength of reinforced concrete columns were made in 1906 by H. B. McFarland and E. V. Johnson at the Chicago laboratory of the National Fireproofing Company.

The columns, three in number, about 10½ in. square and 12 ft. long, were made of 1 : 2 : 4 limestone concrete, and reinforced with ⅜-in. rods placed near the corners. Two columns were tested in compression at normal temperature at ages of two months and three and a half months respectively, and sections 5 ft. to 6 ft. long were cut from them outside of the region of failure for use in the fire test. One of these specimens was covered with 3 in. of solid porous clay tile; the other was tested unprotected, and after the fire test subjected to water application. The third column was cut in two, one part for use in the fire test and the other for a comparable compression test at normal temperature. The age of all columns was 23 months at the time of the fire test.

The three specimens were placed on end in a wood-fired furnace and subjected under no load to a 3-hr. fire test. Furnace temperatures, as indicated by a Bristol thermo-electric pyrometer, ranged from 800 deg. to 1,000 deg. C. for the greater portion of the period. On the day following the fire test they were tested in compression.

The section protected by clay tile was little affected by the test, and developed a compressive strength of 3,127 lb. per sq. in. An 18-in. long specimen cut from the same column, but not subjected to fire test, developed 3,558 lb. per sq. in. The specimen to which water was applied after the fire test failed in the compression test at 674 lb. per sq. in. The column from which it was cut had developed 2,116 lb. per sq. in. at an age of 3½ months, the fire and water treatment having apparently caused a decided decrease in strength. A similar effect was indicated in the case of the third specimen, its strength being 711 lb. per sq. in., against 2,565 lb. per sq. in. for the section of the same column that was not exposed to fire.

The large reductions in strength sustained by the unprotected columns can be ascribed in part to their small size, larger columns being subject to smaller percentage reduction in strength due to surface damage from fire exposure.

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[1] Mittheilungen aus den Mech. Tech. Lab. d. k. Tech. Hochschule, Munchen, Heft 12, 1885; Heft 15, 1887.

[2] *Engineering News*, Vol. 56, p. 316, September 20, 1906.

GERMAN FIRE TESTS ON REINFORCED CONCRETE HOUSES.[1]

The crushing strengths of the various kinds of concrete were determined before and after the fire tests in order to ascertain the average loss of strength. Some of these measurements were made on cubes prepared from pieces cut out of the walls before and after the fires. Others were made on 20 cm. cubes prepared in iron moulds from the mixtures used in the construction of the two houses. Some of these cubes were placed in the houses with alloys to indicate the temperatures reached during the tests. The average figures obtained are summarised below, those in parentheses being unreliable on account of discrepancies between the individual results from which they are calculated.

CRUSHING STRENGTHS OF CONCRETE BEFORE AND AFTER THE FIRE TESTS.

Aggregates of Concretes Tested.	Crushing strength in kilos per sq. cm.		
	Not fired.		Fired.
	70 days old.	16 mths. old.	16 mths. old.
Basalt: from wall, 1 : 4 : 4	138	(119)	112
Basalt: cubes, 1 : 4 : 4	137	(186)	204
Basalt: cubes, 1 : 4	182	207	266
Granite: from wall, 1 : 4 : 4	308	282	225
Granite: cubes, 1 : 4 : 4	213	237	234
Granite: cubes, 1 : 4	206	238	173
Gravel: from wall, 1 : 4	295	326	249
Gravel: cubes, 1 : 4	276	288	221
Pumice: cubes, 1 : 5	57	57	30
Blast furnace slag: cubes, 1 : 5	185	205	123

The above results are not all strictly comparable, since the temperatures to which the cubes were exposed varied from 93 deg. C. (granite, gravel, and pumice) to 230 deg. C. (slag), and 412 deg. C. (basalt). As recorded, however, the figures give the percentage loss of strength on an average as 47 per cent. for pumice, 40 per cent. for slag, 23 per cent. for gravel, 17 per cent. for granite, and for basalt, curiously enough, an increase.

FIRE TESTS OF COLUMNS.[1]

One length of each of the hooped reinforced concrete columns about 3 ft. long was cut outside of the failure region in the fire test and subsequently tested in compression. The limestone concrete specimen sustained a total load of 517,000 lb. as against 243,000 lb. immediately following the fire test, and the trap rock concrete specimen 342,000 lb., compared with 163,000 lb. at the end of the fire test. While a portion of the difference may be due to initial variations in the strength of the concrete, the greater part can be ascribed to recovery in strength of concrete and reinforcement. (Note that the columns were originally designed for a working load of approximately 100,000 lb.)

TESTS BY PROFESSOR F. C. LEA.[2]

Breaking Strength of Cement.—Tests were made on 2 in. cubes and briquettes with 1 in. waist. During heating, cracks began to appear on the surfaces of the specimens at 400 deg. C., and these appeared to increase on cooling. The cracking of the tension specimens was most pronounced at the haunches and on the compression specimens at the corners. The specimens were broken immediately they were cool enough to be handled. The tensile specimens all failed at the cracks at the haunches. The "hardness" of the cement was not affected at temperatures below 600 deg. C. Complete dehydration of the cement does not certainly take place below this temperature. The strength appears to increase up to a certain point, probably due to the "ageing" effect of temperature. The diminution in strength appeared to depend more upon cracks than upon the diminution of the strength of the cohering materials. Subsequent tests indicated that by slowly cooling in the furnace the strength after heating beyond 500 deg. C. was greater than as above.

Breaking Strength of Concrete.—Tests were made on $4\frac{1}{2}$-in. diam. cylinders of concrete made of A.P. cement, Leighton Buzzard sand, and gravel from Water Orton. Heating was done in an electrically-heated furnace.[1] Cracks commenced on specimens heated for 1 and 2 hours at 430 deg. C. On cooling the cracks became more noticeable. At 690 deg. C. the crushing strength of the concrete is only 20 per cent. of its original strength. Up to about 450 deg. C. the concrete appears to be little affected by the heating, and above this temperature the falling off in strength appears to be largely due to loss of bond between the materials. Blocks heated to 430 deg. C. and tested while hot were stronger than other blocks heated to the same temperature and allowed to cool. The number of these tests is not sufficient to prove anything definite, but they do probably indicate that loss of strength is very largely due to cracks that develop further as the specimen cools. The loss of weight of the blocks was quite small. After heating to temperatures higher than 430 deg. C. the pebbles could be picked out of the mass clean. A very large number of buildings have been built in various parts of the country, using gravel in the large aggregate, and even these preliminary tests indicate, what experience has proved in more than one case, that many so-called fire-proof buildings are so more in name than in fact.

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[1] Full Report by Dr. Arthur Holmes in *Concrete and Constructional Engineering*, Vol. 15, Nos. 5, 6 and 7, 1920.

[1] Conducted at Underwriters' Laboratories, Chicago, Illinois, U.S.A., 1917-1919.

[2] *Engineering*, August 27, 1920, Vol. 110, p. 294.

CONDUCTIVITY OF CONCRETE.

Apart from the disintegration of concrete caused by expansion of aggregates on heating there is another point to be considered. Even if the concrete is capable of keeping in place and acting as a protection to the steel it is quite possible that the steel may reach the dangerous temperature owing to the conduction of heat through the protecting coat. Professor Lea carried out a few experiments in order to get some information on this point.[2]

If there is any free water in concrete, or the aggregate is such that chemical changes take place when certain temperatures are reached, or if one aggregate is a better non-conductor than another, then the rate of growth of temperature from the heated surface towards the centre of the element of the structure may be very different in one case from another.

TEMPERATURE GRADIENT ON CONCRETE CYLINDERS. HEATED FROM A FLAT SURFACE.

I.—1 : 2 : 4 gravel
(8·4 per cent. water by wt.)

	Time.	Temp. of Plate.	Temp. (1½ in.)
1st Heating . .	1½ hrs.	730° C.	350° C.
2nd Heating . .	1½ hrs.	730° C.	345° C.

II.—1 : 2 : 4 Rowley Rag.
(8·4 per cent. water by wt.)

	Time.	Temp. of Plate.	Temp. (1½ in.)
1st Heating . .	2 hrs.	755° C.	375° C.
2nd Heating . .	2 hrs.	755° C.	425° C.

III.—Neat Cement.
(24 per cent. water by wt.)

	Time.	Temp. of Plate.	Temp. (1½ in.)
1st Heating . .	2 hrs.	475° C.	100° C.
2nd Heating . .	2 hrs.	520° C.	200° C.

It is interesting to note in all cases the rate of rise of temperature of all the couples up to a temperature of 100 deg. C. was less than after 100 deg. C. had been passed, indicating that there was a little free water in all the specimens. There appeared little evidence of dehydration of the cement.

The results of these tests, though by no means conclusive, indicate that the usual thickness of covering for the steel is by no means sufficient to prevent the dangerous temperature of 650 deg. C. for the steel being reached in those cases where the fire can maintain a high temperature for several hours, and, as far as the tests go, they appear to indicate that not a great deal of difference is to be expected by varying the aggregate, unless materials are used which change chemically below 650 deg. C.

Professor I. H. Woolson[1] carried out some experiments several years ago, and found that when subjected to a temperature of 1,500 deg. F. for 2 hours a point

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[1] Electric heating may possibly produce different results from those produced by hot gases.

[2] *Engineering*, August 27, 1920, Vol. 110, p. 294.

Proc. Amer. Soc. Test. Mat., Vol. 7, 1907, p. 408.

2 in. from the exposed surface of a block of concrete has its temperature raised between 500 deg. F. and 700 deg. F. We learn also that dehydration of the cement crystals begins about 500 deg. F. and is complete at about 900 deg. F. Thus we may reason that dehydration has reached a point 2 in. below the surface of a piece of concrete after being subjected to a temperature of 1,500 deg. F. for 2 hours.

In previous issues of *Concrete and Constructional Engineering* Mr. Arthur Holmes, D.Sc., described the German fire-tests on reinforced concrete houses,[1] and a certain amount of information was there given relating to the penetration of heat in the concrete walls.

The distribution and variation of temperature in the basalt-concrete and granite-concrete walls were found from measurements made at different depths from their inner surfaces and at different times during the tests. The results are fully discussed, but it will be sufficient here to give the average results obtained after 60 and 100 minutes in each case, in order to make a simple comparison between the temperatures to which the two materials were subjected. It is clear from the maximum temperatures attained and the heat penetration that despite its bad behaviour the granite concrete was not heated to the same extent as the basalt-concrete. The figures show, moreover, that the latter did not conduct heat as well as the granite concrete up to 3 or 4 cm., though at 5 cm. the conductivities were practically equal.

SUMMARY OF HEAT-PENETRATION DURING LOWER FIRE TESTS.

Depth from Inner Surfaces of Walls.	Temp. after 60 mins. (average).	Temp. after 100 mins. (average).
Basalt :		
1 cm.	312° C.	377° C.
3 cm.	204° C.	282° C.
5 cm.	165° C.	211° C.
Granite ;		
1 cm.	252° C.	316° C.
3 cm.	181° C.	258° C.
5 cm.	122° C.	179° C.

HEAT CONDUCTIVITY TESTS BY THE B.F.P.C.

The slabs tested were of plain concrete, 3 ft. 6 in. square and 5½ in. thick, and the following conclusions are taken from B.F.P.C. Red Book No. 252.

The results show that the various concrete aggregates are very much alike in their heat and temperature conducting properties, and that an approximate rule for the temperature distribution in the lower 2 in. of slabs of the aggregates tested when their lower surface has been exposed to fire for two to four hours is :—

Deduct from the temperature of the flames in contact with the surface 300 *deg. F.* (148·8 *deg. C.*) for ½ *in.* (0·013 *m.*), 500 *deg. F.* (260·0 *deg. C.*) *for* 1 *in.* (0·025 *m.*), *and* 900 *deg. F.* (482·2 *deg. C.*), *for* 2 *in.* (0·051 *m.*) *within the slab.*

The temperatures are approximately 100 deg. F. (37.7 deg. C.) higher in slabs composed of flint or certain types of clinker and coke breeze, and lower in slabs composed of limestone or burnt Gault clay.

The temperature records throughout the slabs show that reinforcing rods with only ½ in. (0·013 m.) cover to the lower layer will in all types of concrete tested be raised to 1,200 deg. F. (648·8 deg. C.) in under 4 hours by a fire underneath of the standard type used in these tests. At this temperature the rods will break under the working load of 16,000 lb. per sq. in.

A cover of 1 in. (0·025 m.) to the lower layer of reinforcing rods of concretes containing limestone or Gault clay aggregates protects the reinforcing rods from being raised to 1,200 deg. F. (648·8 deg. C.) but other aggregates do not afford the necessary protection.

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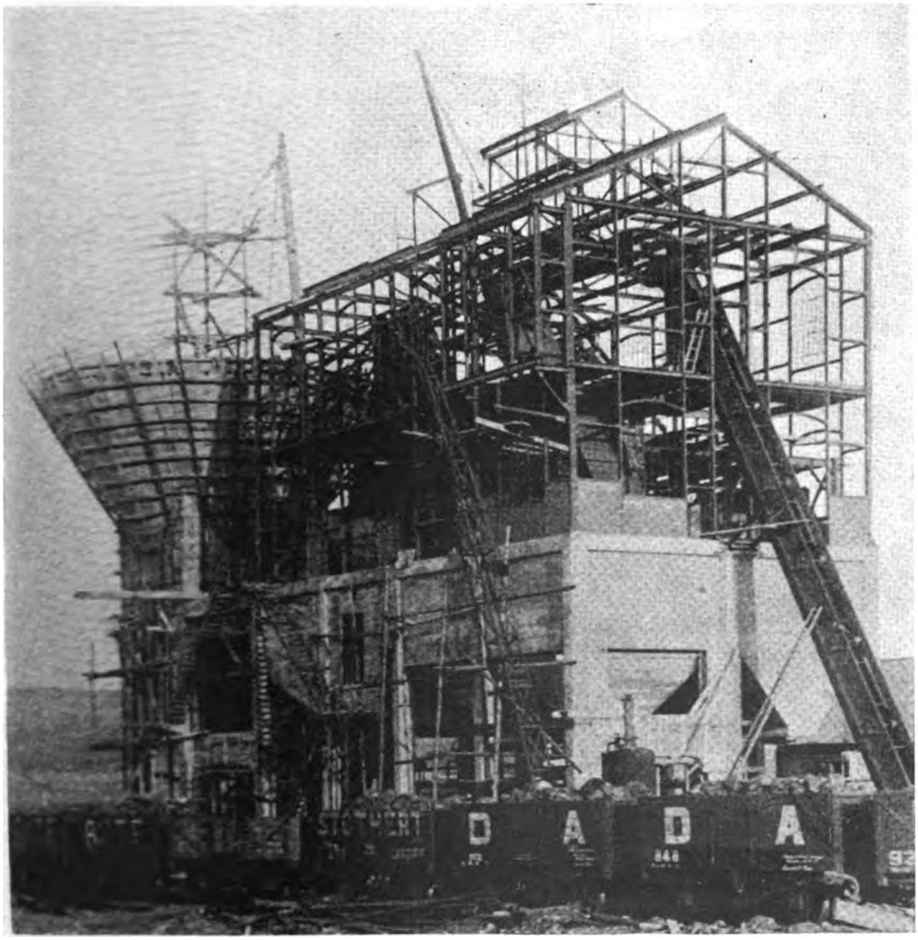
[1] *Concrete and Constructional Engineering*, Vol. 15, Nos. 5, 6, 7, 1920.

REINFORCED CONCRETE COAL WASHING PLANT.

THE building illustrated by the photographs is a British "Baum" coal washery, of 100 tons per hour capacity, at the Tower Collieries, Hirwain. This is one of Sir D. R. Llewellyn's numerous collieries, and it is by his courtesy that we are enabled to publish the accompanying photographs and description.

Owing to the nature of the ground, both building and tank are founded on a reinforced concrete raft. The bunkers, sumps, and settling tank are entirely in reinforced concrete, while the upper portion or housing for the coal washing machinery is carried out in steel framework and brick panelling. This combination of building materials produces a very pleasing effect, while at the same time the use of constructional steelwork in the upper portion considerably expedites the completion of the building.

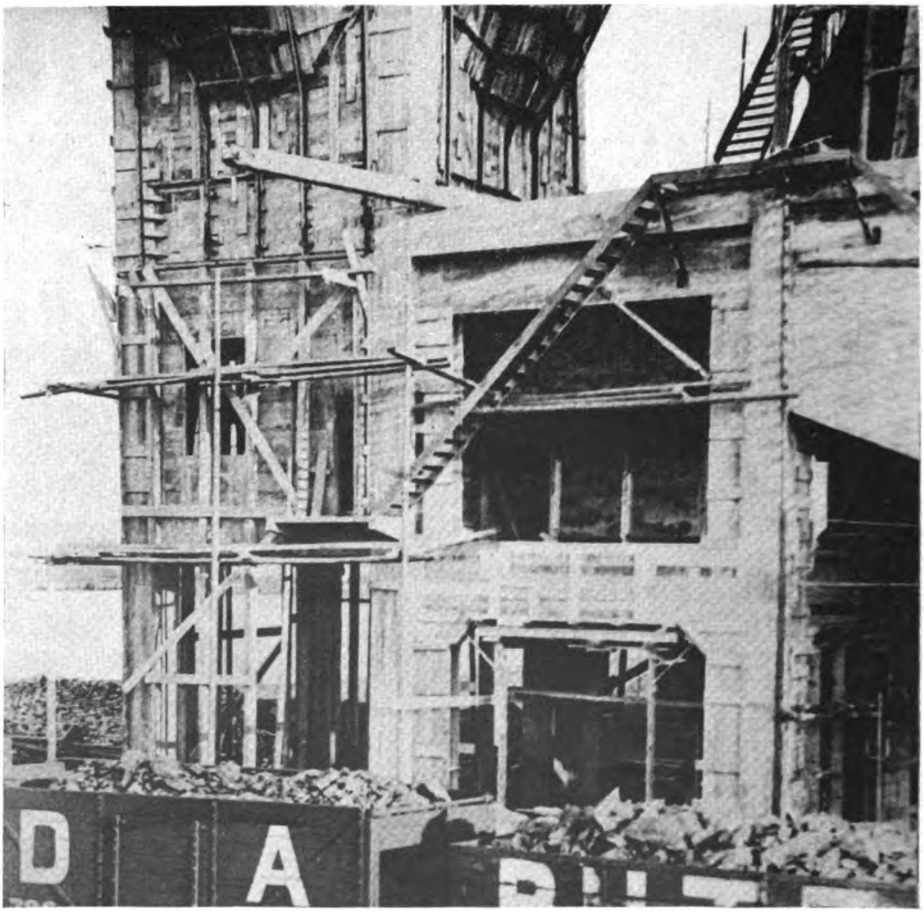
Concreting was commenced on this job on April 12, 1922, and it is expected that the plant will be working shortly. The whole of the concrete has been placed by means of a mast hoist chuting plant, and the work generally is designed and carried out with special attention to the reduction of unnecessary labour on the site consistent with a first-class job.



REINFORCED CONCRETE COAL WASHING PLANT AT HIRWAIN.

Sir D. R. Llewellyn, Bart., decided to install a washery of this type so that the small coal despatched from the colliery should be of the best possible quality. The object of these washeries is to remove the dirt, shale, or stone from the small coal, thus improving its quality. All practical coal washeries depend upon the difference in specific gravity between the coal and the dirt for the elimination of the latter, other methods having been found impracticable on a commercial scale. In the British " Baum " washery here illustrated, the coal, after being tipped from wagons by electric tippers, is elevated to pass through a large tank containing water. This water is caused to pulsate, which motion lifts the coal away from the dirt and enables the coal to be taken off from the upper portion of the box or tank, the dirt passing away separately for disposal on to a tip. The coal is then classified into various sizes for sales purposes, and is stored in the hoppers as shown for loading into wagons when required. The water for the washing is stored and clarified in the large conical tank shown in the illustration, the clean water passing by gravity from the top of the tank through the plant and being pumped back again after use. The fine coal settling from the water passes by gravity from the bottom of the tank over a set of vibrating sieves for drying purposes, and thence into one of the hoppers.

The fundamental principles of the



Reinforced Concrete Coal Washing Plant at Hirwain.

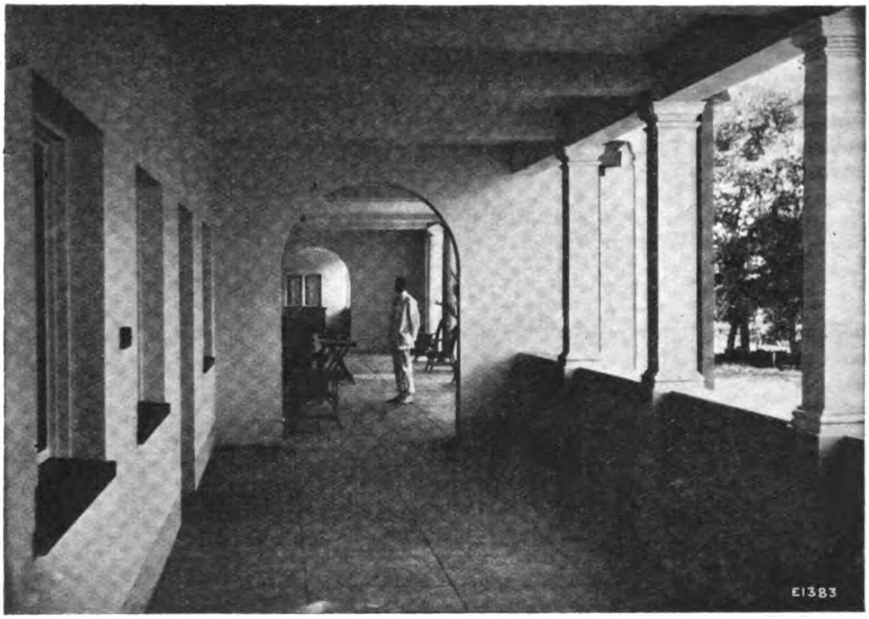
British "Baum" coal washing plant are:—

(1) The necessary pulsation of the water in the washing boxes is produced by compressed air, thus allowing a good motion for the separation of the coal and enabling the coal to be washed before classification, which is most advantageous.

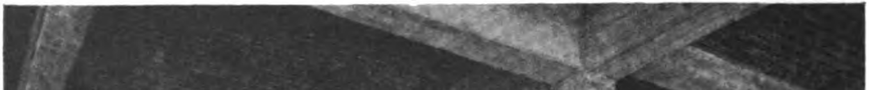
(2) Owing to the conical tank and the water circulation system there is no effluent from the plant apart from the dirt, which is taken to a tip, and settling ponds are non-existent.

Messrs. Simon-Carves, of Manchester, carried out the whole of the design and erection of the reinforced concrete, machinery, and building work for the plant; at the present time this firm has fifteen similar plants, of capacities varying from 50 to 150 tons per hour, under construction in this country.

Reinforced Concrete Coal Washing Plant at Hirwain.



THE VERANDAH.



SHOWING CONSTRUCTION.

VICTORIA FALLS HOTEL, RHODESIA. (*See p.* 83.)

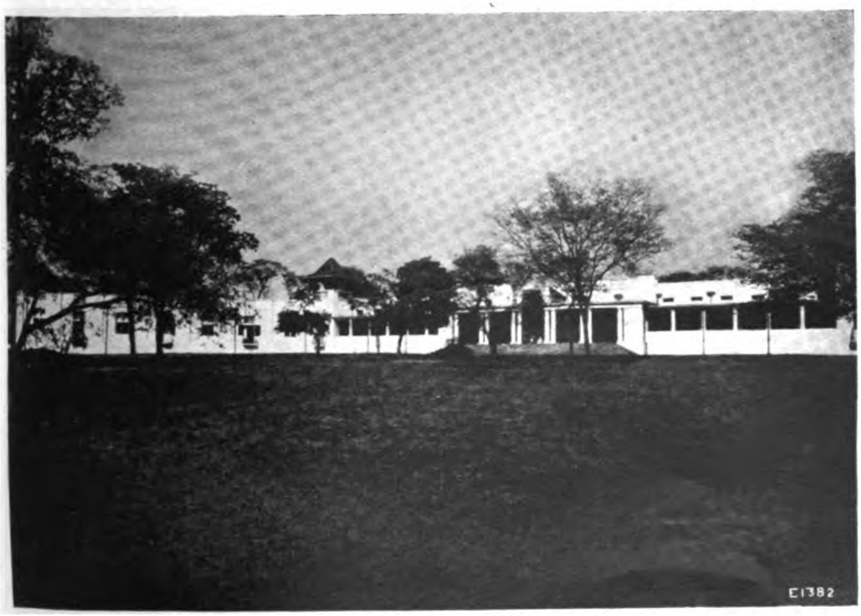
THE VICTORIA FALLS HOTEL, RHODESIA.

THE new hotel at Victoria Falls, Rhodesia, is an interesting example of the use of reinforced concrete. The front elevation of the hotel faces the gorge and has an excellent view of the Falls. The centre portion of the building has a frontage of about 200 ft., and is devoted mainly to a lounge hall, measuring 60 ft. by 35 ft., reception rooms, drawing and writing rooms, loggias, and private suites. The two side wings have a frontage of 100 ft. each, and consist of bedrooms, bathrooms, etc. The dining room measures 60 ft. by 37 ft., and is capable of seating one hundred persons. It is situated at the back of the main building, and the kitchen quarters adjoin it.

The building is constructed of a skeleton of reinforced concrete columns and beams, the filling above ground level being brickwork carried on reinforced concrete beams. The ground floor varies from about 2 ft. to 8 ft. above the natural level of the ground, and the whole building is surrounded by a reinforced concrete retaining wall to support the filling upon which the ground floor is carried. The whole of the floor slabs are reinforced with 26-gauge "Hy-Rib," the material acting as centering and reinforcement combined. The spans of the slabs vary from 5 ft. to 11 ft., and the thickness from 3 ft. to 5 ft. The beams, columns, bases, and retaining wall are reinforced with "Kahn Trussed" and "Kahn Rib" bars, the largest span of the beams being 37 ft. The partitions vary from 11 ft. 6 in. to 16 ft. high, and are constructed of "Hy-Rib" plastered to a thickness of 3 in. Approximately 100 tons of reinforcement and 100,000 sq. ft. of "Hy-Rib" are incorporated in the building.

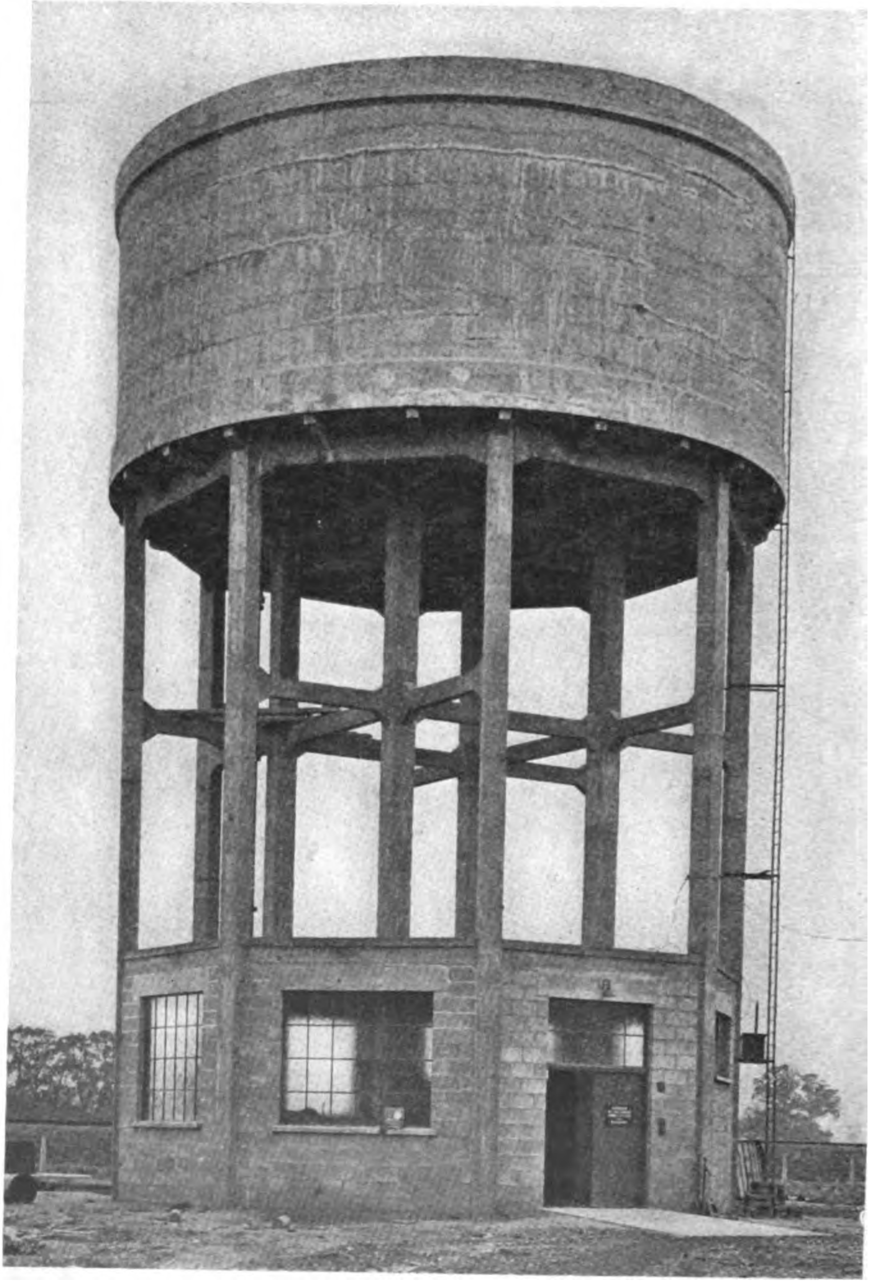
Mr. Frank Scott, M.S.A., Bulawayo, was the architect.

The whole of the reinforced concrete was designed by the General Fire Appliances Co., Ltd., agents for the Trussed Concrete Steel Co., in accordance with the architect's plans. The contractors for the work were Messrs. Sellick & Co., Bulawayo, and the building took twenty-seven months to complete.



[Mr. Frank Scott, M.S.A., Architect.

VICTORIA FALLS HOTEL, RHODESIA.



New Water Tower at Aylesford. (*See p.* 85.)

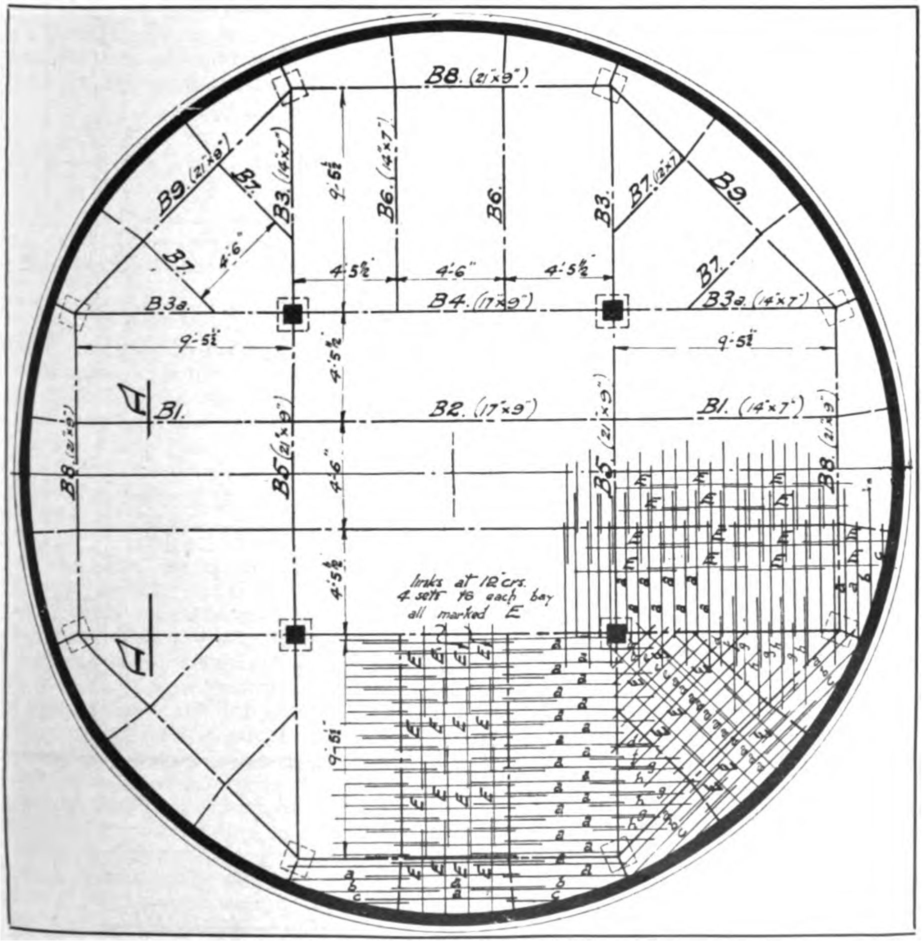
WATER TOWER AT AYLESFORD.

We illustrate herewith a water tower just completed at Aylesford for Messrs. Albert E. Reed & Co., Ltd., paper manufacturers, of Cannon-street, London.

In the first place it was requested that the design should be plain and at the same time have a good appearance, and since completion the proprietors have stated that the design and general appearance are quite satisfactory, from both the æsthetic and service point of view.

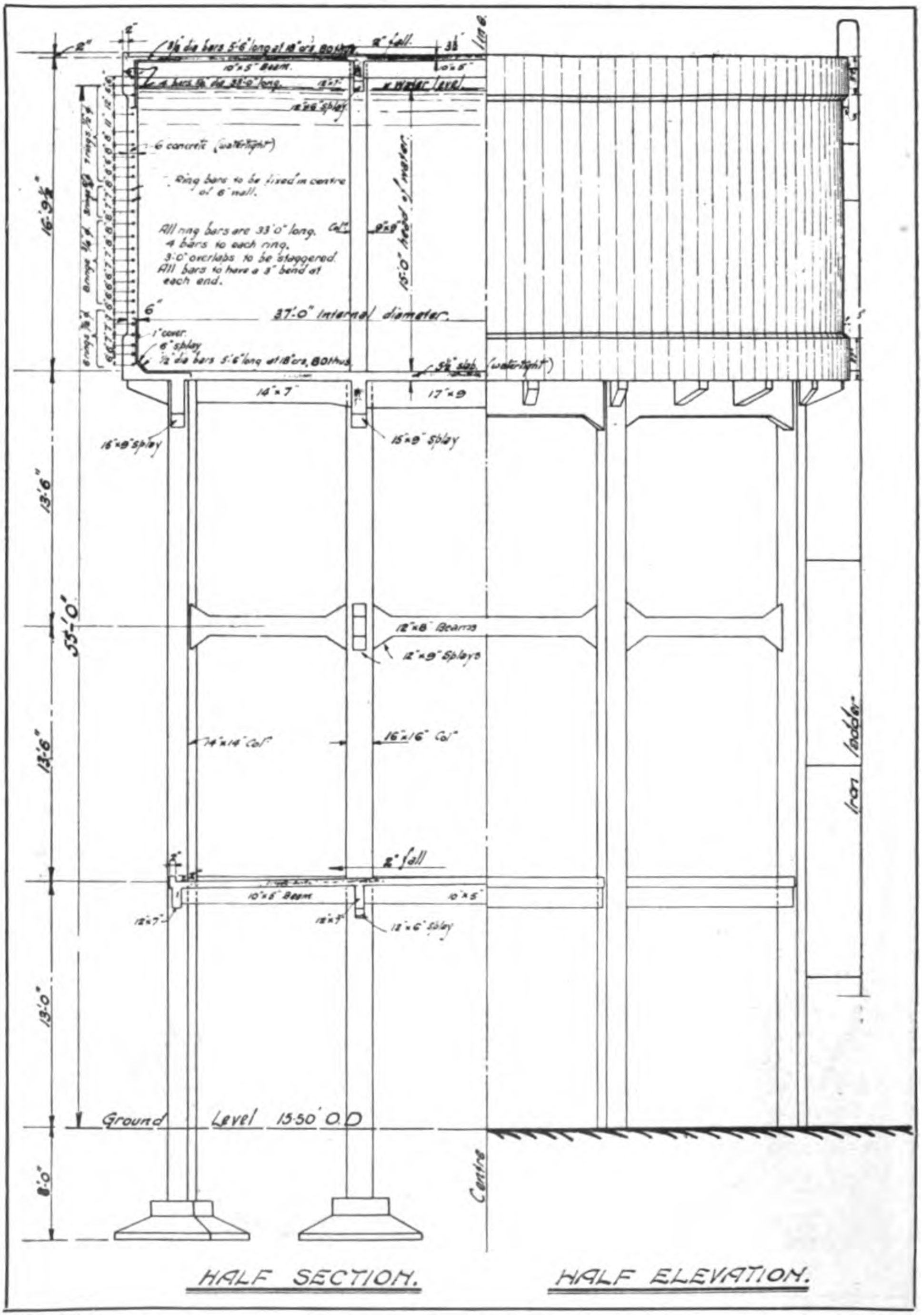
The tower has a capacity of 100,000 gallons, with a total head of water of 55 ft., the container itself being 37 ft. internal diameter, and 16 ft. 6 in. deep, with the water level at 15 ft. to give the required capacity. The floor of the tank is 5½ in. thick supported on beams 14 in. by 7 in., 17 in. by 9 in. and 21 in. by 9 in. The wall thickness is 6 in., with a top and bottom cornice 24 in. by 3 in. The roof slab is 3½ in. concrete on 10 in. by 5 in. and 12 in. by 7 in. beams; a manhole 2 ft. square is left in a suitable position opposite the ladder.

The tank is supported on twelve columns, eight of which are 14 in. by 14 in. and form an exterior ring at 35 ft. diameter centres, while the four interior columns are 16 in. by 16 in., at 13 ft. 5 in. centres.



Showing Lay-out of Slab Bars and Slab Links.

Reinforced Concrete Water Tower at Aylesford.



New Water Tower at Aylesford.

There is one set of braces to columns between the tank floor and the transformer house roof; these are 12 in. by 8 in. section, and are splayed at each connection to the column.

From ground level up to 13 ft. high the exterior columns have been housed round with concrete blocks to form a pump house, the concrete roof to which acts as a further bracing to the columns. A 6-in. reinforced concrete floor laid on the ground forms the floor to the pump house. The column foundations were taken down to a level of 8 ft. below ground, and a load of 2 tons per sq. ft. imposed on a good ballast stratum.

An iron ladder is fixed in the position shown in the illustration, and supported by W.I. straps to the columns and tank.

It is interesting to note with regard to the cost of this tower that estimates were received for the construction of a cast-iron tank on steel legs, and also for a pressed steel tank; the result showed reinforced concrete to be considerably cheaper than the cast-iron scheme and approximately the same as the pressed steel. The advantages claimed for concrete of durability and minimum cost of maintenance alone were sufficient to decide the contract.

The whole of the concrete work is reinforced with "Keedon" and lattice reinforcement, and was carried out under the general supervision of Mr. Bonny, the Company's Resident Engineer. The contractors were Messrs. Griggs & Sons, concrete specialists, of 100, Victoria Street, London, S.W.1, who worked to the plans of Messrs. Johnson's Reinforced Concrete Engineering Co., Ltd., who were responsible for the design and supplied the whole of the reinforcement.

BOOK REVIEWS.

The Load-bearing Power of Piles. By Dr. H. Dörr.

(68 pp., with 61 illustrations. Berlin: Wilhelm Ernst and Sohn. Price 270 marks.)

Many attempts have been made to ascertain the maximum load-bearing power of piles under various conditions, but the results have not been satisfactory. The author has followed the recent work of Stern and Kafkas, who omitted to consider several important factors. The author also bases his new formulæ on Engesser's "wedge" theory of equilibrium, and by these means develops a number of formulæ for piles of various shapes. Special consideration is given to the resistance of the piles to compression and tension, to the maximum length of the piles, to the pressure of the structures they support, and the effect of the nature of the ground in which the piles are immersed and its co-efficient of friction. Some particularly useful notes are given on the effect of various methods of ramming, on the sinking of the piles under load, and on the spacing of the piles in any given area. The risk of the piles bending is briefly considered.

The chief conclusion reached by the author is that all previous formulæ for ramming piles are of no value and are apt to be misleading, because they do not take sufficiently into account the specific properties of the ground to be piled and the enormous increase in carrying power obtained by a short increase in the length of a pile; in one case, the increase in the length of a pile from 16 ft. 6 in. to 20 ft. (i.e., an addition of only 3 ft 6 in.), raised the load-bearing power from 20 tons to 31·4 tons, or an increase of 57 per cent., the lower end of each pile being in loose silt. Another important conclusion is that conical piles have a greater carrying power than those with parallel sides, but it is merely a waste of material to make the piles conical throughout; the cone should extend from the base to a height which depends on the nature of the ground; above this point, the pile may be of smaller diameter, and have parallel sides. A straight pile with an enlarged foot of the shape of an inverted cone is only of use, however, when it rests on solid ground. If driven into quicksand the loss of frictional resistance in the pile as a whole may be greater than that of a slightly tapered pile of the same length and upper diameter. The author regards a metal sheathing to the pile as undesirable, and the rougher the surface of the piles the greater will be their resistance to slipping when once they have been rammed into place. He regards longitudinal reinforcement as useless when piles are merely subject to load, but admits its value near the head of the pile. He is greatly in favour of piles produced *in situ*, and claims that his investigations show their superiority over rammed piles. Apart from this, the author's investigations confirm those of Bernhard, and show that the manner in which the pile is driven is a matter of small importance.

No less than thirty-four examples of different types are fully worked out, and as there is an illustrative diagram on every page the book is easily followed by any reader who is sufficiently acquainted with the German language.

The Economics of Baths and Washhouses Design. By John A. Davenport, M.Sc., A.M.Inst.C.E.

(28 pp. Liverpool: Littlebury Bros. Price 2s.)

This little book, which within its limited space goes very thoroughly into the methods of laying-out and arranging public baths and washhouses, should be of considerable value to Borough Engineers and others responsible for such undertakings. As the title indicates, the author is more concerned with the economic aspect of the question, i.e., so arranging the different departments that the minimum of labour is required for running the establishment, and installing plant of a size which can most economically deal with the demand for hot water with the least amount of idle boilers in the winter when the requirements slacken. The acutal construction of the buildings is not touched upon except in the introduction, where the author, after a brief reference to "the superiority of reinforced concrete for baths and washhouses work" on account of its durability, mentions the uses of concrete tanks for holding boiling water. "Quite recently," he says, "a large wrought-iron tank . . . has been lined with two inches of reinforced concrete and is now in commission full of boiling water. This method of repair saved the disturbance and disorganisation attendant upon the taking down of an old tank and replacing a new one in the roof of a building, quite apart from the direct savings."

Concrete Bridges. Part II. Arches. By C. Kersten, Docent at the Building School, Berlin.

(228 pp.; 521 illustrations, 4th edition, revised. Berlin: Wilhelm Ernst and Sohn. Price, in paper, 297 marks; in cloth, 360 marks.)

This is one volume in a series on concrete bridges, and is intended alike for the student and constructional engineer. It summarises, with characteristic German thoroughness, the various theories and the general state of knowledge of the art of bridge construction in concrete, and brings these into a handy compass. The book is in two parts, the first of which deals with the construction of bridges with as much freedom as possible from mathematical considerations. The second portion of the book deals with the theoretical calculations, to a limited extent but with ample examples.

The volume is in the German language, but readers familiar therewith will find the descriptions are very lucid, and the abundance of illustrations adds greatly to the ease with which the author's descriptions may be followed. An additional value is given to the volume by the fact that the author is as conservative in his ideas as he is extensive in his reading. The chief defect of the book is the absence of an index, which makes reference to any particular method of construction a tedious task. A minor omission is that of the composition of the concrete to be used in the various bridges. The amount and arrangement of the reinforcement, as well as the design of the bridges, are described in full detail, but only the barest reference is made to the concrete itself. Notwithstanding these omissions, the work is one which—if it were translated into English—would be of great value to all engaged in the construction of arched bridges.

THE SLENDERNESS-RATIO AND STRENGTH OF CONCRETE COLUMNS.

THE following is an abstract of an article by Mr. F. E. Giesecke (Head of the Engineering Research Division, University of Texas) in a recent issue of *Engineering News-Record*, to whom we express our acknowledgments :—

Various regulations or formulas to express the relation between strength and slenderness-ratio of reinforced-concrete columns are in current use. The most important ones known to the writer fix the relation between allowable load on long columns (P') and on short columns (P) as follows :

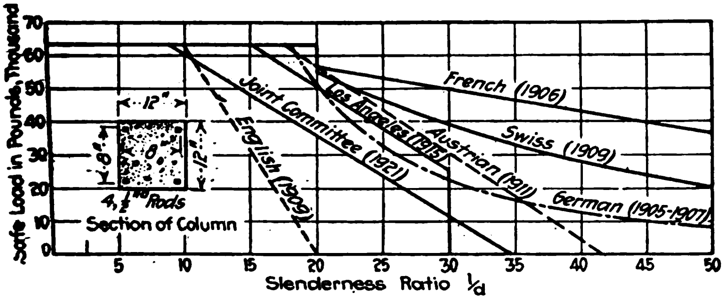


FIG. 1. SAFE LOADS ON 12 IN. BY 12 IN. CONCRETE COLUMNS UNDER DIFFERENT REGULATIONS.
[Computed for Equal Base Stress.]

(1) French regulations of 1906, for l/d greater than 20,

$$P' = \frac{P}{1 + \frac{k}{10{,}000}\frac{l^2}{r^2}}$$

in which k is ¼ for square-ended columns and 1 for round-ended columns.

(2) German regulations of 1905 and 1907, for l/d greater than 18,

$$P' = \frac{\pi^2 EI}{s\,l^2}$$

where s, the factor of safety, is 10 in the German regulation.

(3) English regulations of 1909, fix values of P'/P for ratios of virtual length to effective diameter of a square pillar ranging from 15, 18, 21, 24, 27, 30 at 1, 0·8, 0·6, 0·4, 0·2, and zero. The virtual length of the column varies from l to $4l$ depending on end conditions ; the effective diameter is measured to the outside of the outermost vertical reinforcement.

(4) Swiss regulations of 1909, for l/d greater than 20.

$$P' = \frac{45}{35}\,\frac{P}{1 + \left(\frac{l}{100r}\right)^2}$$

(5) Austrian regulations of 1911, for l/r greater than 60,

$$P' = P\left(1{\cdot}72 - 0{\cdot}012\,\frac{l}{r}\right)$$

(6) Los Angeles regulations of 1915, for l/d greater than 15,

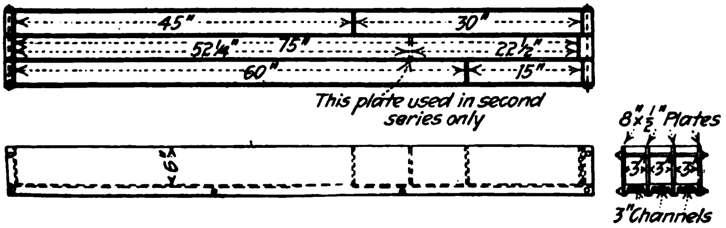


FIG. 2. GANG MOULD FOR 3 IN. BY 6 IN. COLUMNS.
[Upper edges of plates held together by clamps until concrete was set.]

$$P' = P\left(1{\cdot}6 - \frac{l}{25d}\right)$$

(7) Joint Committee, in its tentative specifications of 1921, for l/r greater than 40,

$$P' = P\left(1{\cdot}33 - \frac{l}{120R}\right)$$

where R is the least radius of gyration of the core.

In order to be able to compare these six formulas, the strengths of 12-in. square columns of various lengths, reinforced with four ½-in. square bars 7½ in. on centres, were calculated by each of the six formulas for equal loads on short columns. The results, plotted in *Fig.* 1, indicate that the French and Swiss regulations are based on Rankine's formula and the German regulations on Euler's formula, while the Austrian, Los Angeles, Joint Committee, and English rules seem to be straight-line approximations of Euler's formula.

Since there is a considerable variation in the results it was decided to make experimental determination of the strength of plain concrete columns, and a series of tests was begun by the Engineering Research Division of the University of Texas in 1919.

Test Procedure.—The columns tested were 3 in. by 6 in. in cross-section and varied in length from 15 in. to 75 in., so that the slenderness-ratios were 5, 10, 15, 20, and 25. They were made in moulds formed by bolting 8 in. by ½-in. steel plates to 3-in. channels and inserting dividing plates to limit the lengths of the columns (*Fig.* 2). The columns were thus poured from their sides. As all columns of a series were made at the same time the character of the concrete was exactly the same in any one series.

Eight such series were prepared, differing in mix and consistency. In some of the groups the concrete was rodded; in others it was tamped. The columns were tested at the age of 28 days. The results of this series of tests are shown in *Fig.* 3.

Inspection shows that the strength of these concrete columns varied only slightly with the slenderness-ratio, which, however, did not exceed 25; but it also shows that a variation in the consistency of the mix, or a variation in the degree of richness of the mix, or a variation in the method of placing the concrete may produce a very great variation in strength, more than 100 per cent.

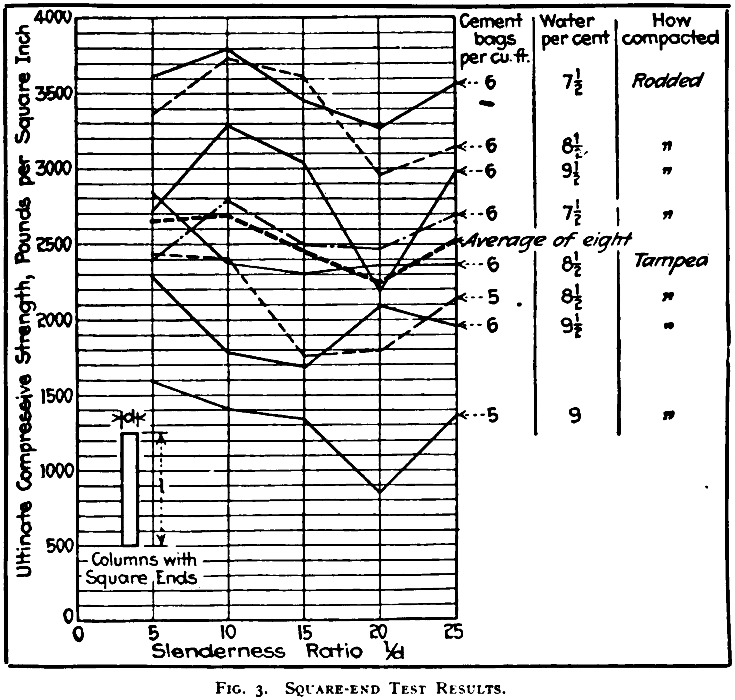


FIG. 3. SQUARE-END TEST RESULTS.

[Each point of the full lines represents one column; the dash line is the average of the eight series. Maximum size of aggregate 1½ in. in Series 1, ¾ in. in others. Reading from top down on right-hand side, series numbers are, 2, 4, 6, 1, average 3, 7, 5 and 8.]

Round-End Tests.—Since these columns showed such a slight variation in strength with slenderness-ratio, it was decided to repeat the experiment on round-end columns between spherical bearings. This method of testing would ensure central loading and would double the virtual length and therefore the slenderness-ratio of each column. For this purpose the bearing blocks shown in *Fig.* 4 were constructed. Since the addition of the two bearing blocks made the longest column of the first group too long for the testing machine, that column was divided into two parts (dotted line in *Fig.* 2). Thus each series of the 1920 group consisted of six round-end columns having slenderness-ratios of 7·34, 9·75, 12·25, 17·17, 19·63, and 22·13, or, for the equivalent square-ended columns, 14·68, 19·5, 24·5, 34·34, 39·26, and 44·26.

For each series one or more 6 in. by 12 in. cylinders were made to identify the type of concrete. Five series of columns were made in the autumn of 1920 and tested during the summer of 1921, at an average age of about 9 months. Each series consisted of five sets of columns, and the entire group consisted of 150 columns. The five series differed from each other in the type of concrete. In all, more mixing water was used than was necessary in order that the experimental columns might be no better than those generally found in practice.

Anomalous Buckling.—When the testing of the columns was begun it was found that the columns bent in the 6-in. direction instead of in the 3-in. direction, as in *Fig.* 5, showing a column 15 in. long; it is evident that this column failed by flexure in the 6-in. direction. This peculiar condition was concluded to be due to the fact that the columns were poured from the side and into fairly tight steel forms, and that therefore the concrete on the upper side of the column contained more mixing water than that in the lower side; the strength effect of water content was sufficient to overcome that of slenderness-ratio.

In order to test the columns satisfactorily it was necessary to apply the load eccentrically about ¼ in., as shown in *Fig.* 4. This degree of eccentricity seemed to be just sufficient to overcome the effect of the variation in the strength of the concrete in the direction of the width of the columns.

Fig. 6 shows the results of the tests of this group. The lengths used in this diagram are the actual lengths of the specimens from centre to centre of the spherical bearings; the vertical lengths are twice as great.

Results.—The diagrams of *Figs.* 3 and 6 show that for the limits of slenderness-ratio examined the strength of the column is almost independent of the slenderness-ratio. But they also show that the results of the tests are not as uniform as they should be in a carefully-conducted investigation. The writer believes that the irregularities in the results are due to a difference in the water content of the concrete, which, in turn, was due to the particular location of the column in the mould and to the location of the mould.

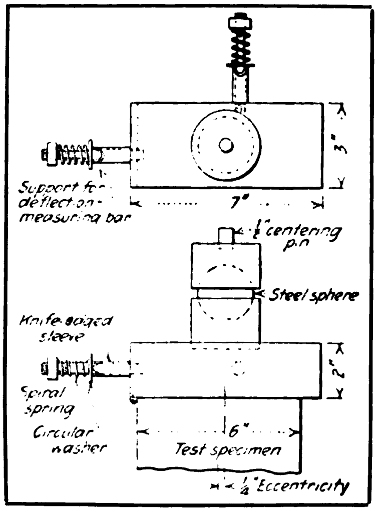


FIG. 4.—END SUPPORT FOR ROUND-END COLUMNS. [Eccentricity adjusted to prevent buckling in 6-in. direction.]

For example, *Fig.* 3 shows that the 30-in. columns were the strongest and the 60-in. columns the weakest of the entire group. If the mould shown in *Fig.* 2 sloped downward in the direction from the upper right to the lower left of the plan, the water which was worked out of the concrete flowed so that the 30-in. columns would contain the least water and the 60-in. columns the most; this would produce the differences in strength shown in the diagram. In the second group (*Fig.* 6) the two centre columns, i.e., the 22½-in. and 52½-in. columns, were the strongest. If the mould was level and the concrete a little thicker in the centre columns than in the side columns the water worked out of the centre columns would flow over the side columns, and as a result the side columns would be weaker than the centre columns.

Another interesting fact developed in

the second series of tests was that the strength of the 6 in. by 12 in. check cylinpers was in several cases somewhat lower than the strength of the slender round-ended columns. This was probably due to the fact that the cylinders, being twice as deep as the columns, retained relatively a higher proportion of the mixing water, and therefore attained a lower strength.

Theoretical Length-Relation.—Considering the column from a theoretical point of view, it is evident that, so long as the column does not bend its strength is independent of its slenderness-ratio, and that after flexure has taken place, either due to non-homogeneity of the material or due to eccentric loading, the load which can be safely supported by the column will decrease as the flexure increases. Since the bending moment in the column at any point is proportional to the deflection at that point, and since the flexure of a bar can be determined by considering the moment diagram as the loading diagram and finding the resulting moment, it can be shown that the deflection of a long column is proportional to the square of its length.

Then the stress due to flexure also varies as l^2 and since Euler's formula gives the ultimate strength of a long column as $\frac{\pi^2 EI}{l^2}$, it is evident that Euler's formula can only be applicable in those cases where the stress due to flexure is infinitely great as compared with the direct stress due to the load. However, in the tests reported above, measurement of the deflections by means of Wiesler dials (*Fig.* 6) showed a maximum deflection of only 0·013 in. This occurred in a round-end column 67 in. long, at a load about 90 per cent. of the ultimate, at which time the deflectometers were removed. The stress due to flexure was therefore 59 lb. per sq. in., and the direct stress was 2,240 lb. per sq. in.

In order that the two stresses might have been equal, the column would have had to deflect about ½ in., or about 38 times as much as it did. It therefore seems clear that the stresses due to flexure in concrete columns will always be very much less than the direct stresses due to the loads and that therefore Euler's formula and the straight-line formulas based on Euler's cannot be correctly applied to concrete columns. The error due to their use is on the side of safety, however.

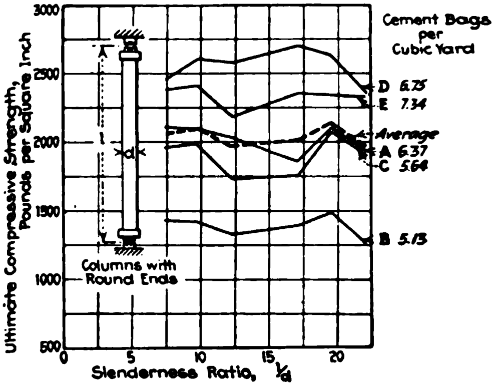


FIG. 7. ROUND-END TEST RESULTS.
[Each point of the full lines represents 5 columns; the dash line is the average of the five tests.]

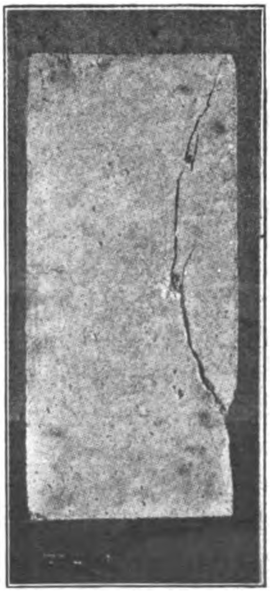


FIG. 5. SHORT TEST COLUMN FRACTURED IN 6-IN. DIRECTION.

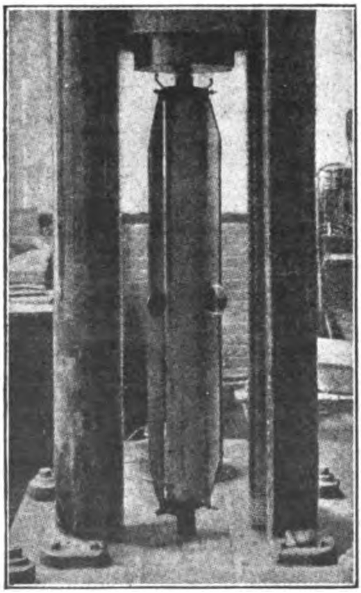


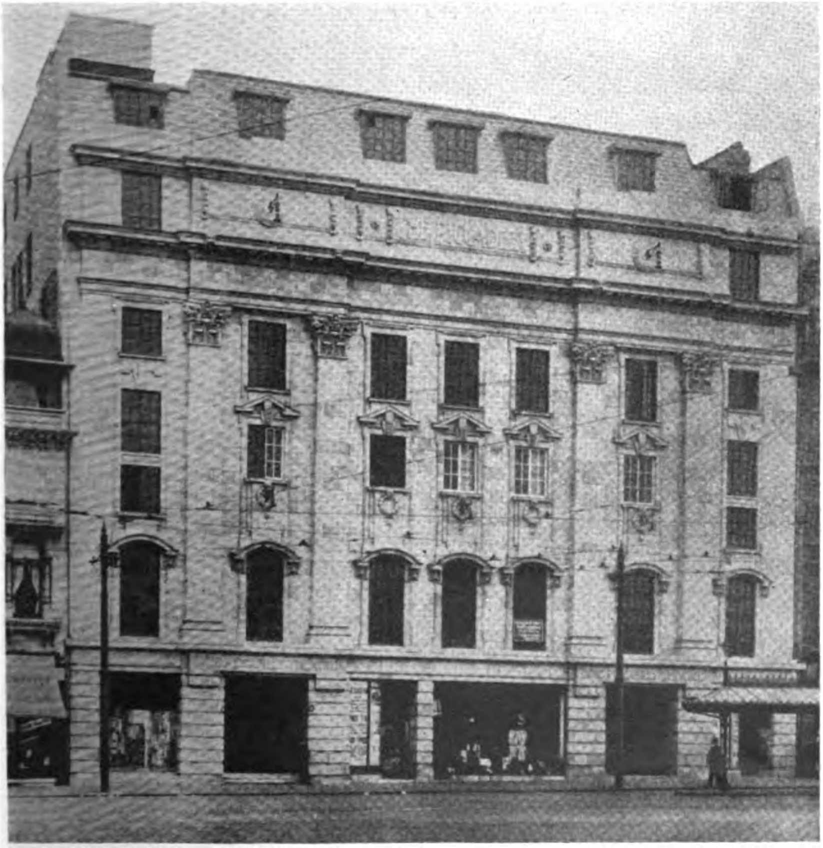
FIG. 6. ROUND-END TEST ARRANGEMENT.
[Two Dials measured Deflection.]

PICCADILLY PICTURE THEATRE, MANCHESTER.

The Piccadilly Picture Theatre occupies a central position in Piccadilly, Manchester. The accommodation provides for approximately 2,400 seats in the cinema and 500 in the café. Electric passenger lifts serve all floors, and ventilating, air-washing, and heating plant changes the air in the theatre six times every hour.

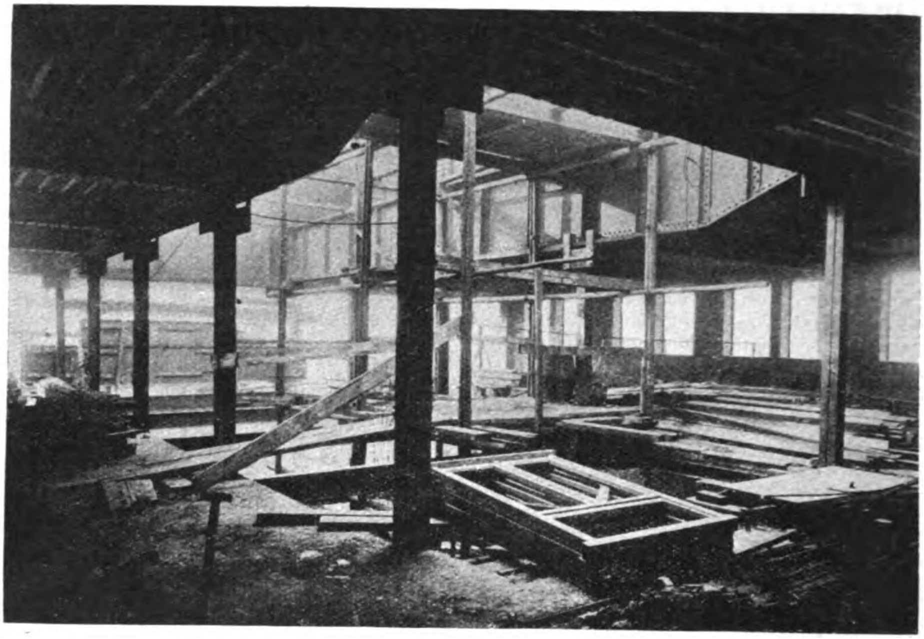
The structure is completely steel framed, the filling to floors, walls, and balconies being in patent slab reinforced concrete. Circumstances helped to make the construction of the building interesting from several standpoints. In the first place, the old Mosley Hotel which occupied the site was not to be removed until the steel skeleton structure was in place. Second, the building was commenced at a period when cinema construction had to be suspended in favour of housing, and this decision completely altered the structural plans. The new plans provided for two floors of hotel or office premises above the cinema, and as this class of work was not prohibited all efforts were concentrated thereon. Later the restriction was withdrawn and work on the theatre commenced in earnest.

As the floors of the Mosley Hotel had to remain temporarily, holes were made through the old flooring and walls, and the four main stanchions threaded through and fixed in position. These stanchions are of the heavy joist compound type, plated throughout their length, and practically carry the whole of the roof and two floors of the hotel

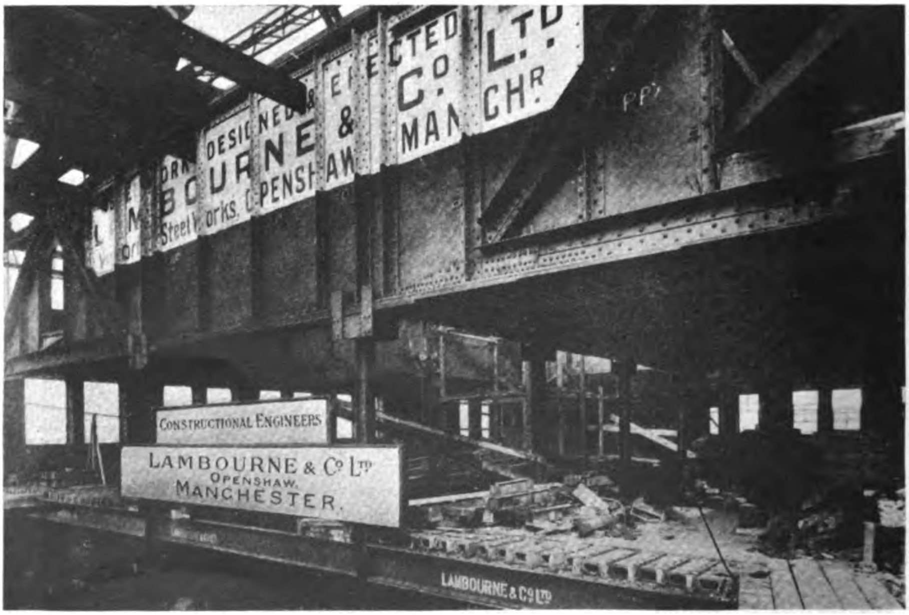


[*Mr. Percy Hothersall, Architect.*

Piccadilly Picture Theatre, Manchester.



Showing Channel Suspenders and Slung Floor (over Theatre), from Inside.



Showing Front Roof Girder (with Door Openings) and Channel Suspenders and Slung Floor over Theatre.

PICCADILLY PICTURE THEATRE, MANCHESTER.

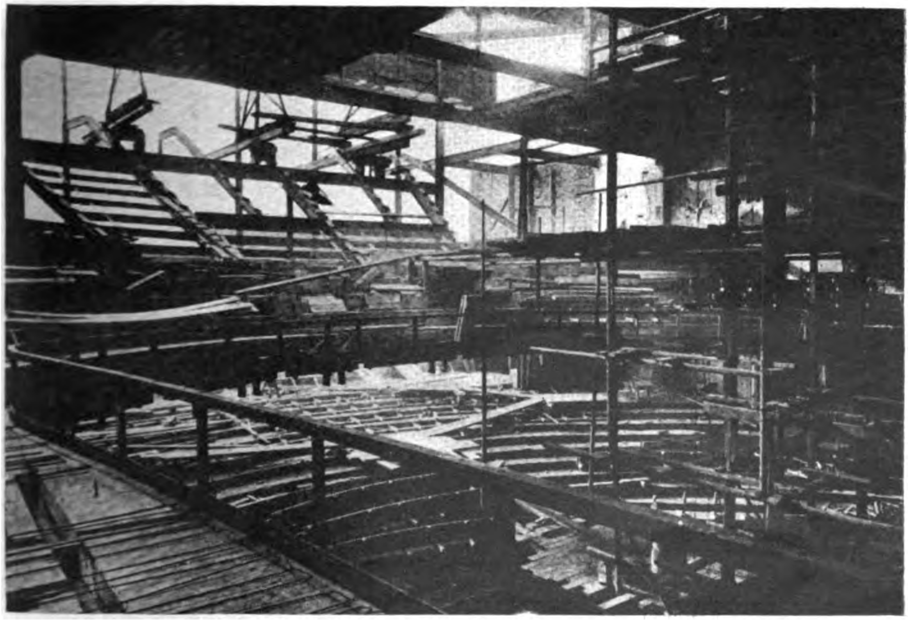
portion, because upon them rest the two main roof girders, approximately 70 tons each and nearly 90 ft. span. The depth of each girder is the height of one floor of the hotel, and doorways are allowed for through the webs near each end. Two secondary girders rest between the two main girders, and from this complete roof frame is slung the whole of the new hotel floors above the theatre; the whole of the structural work was erected from the roof downwards.

Owing to the roof girders being erected first and covered in last they became an interesting public sight for the larger portion of twelve months—hanging, as they apparently were, in mid air over the old Mosley Hotel. The girders were placed in position before any of the old façade of the hotel was removed, and they remained whilst the old building was taken down and until the new edifice rose to meet them and finally cover them in. The suspension members (ten in number) between the roof frame and hotel floors consist of double R.S. channels with special connections at head and base.

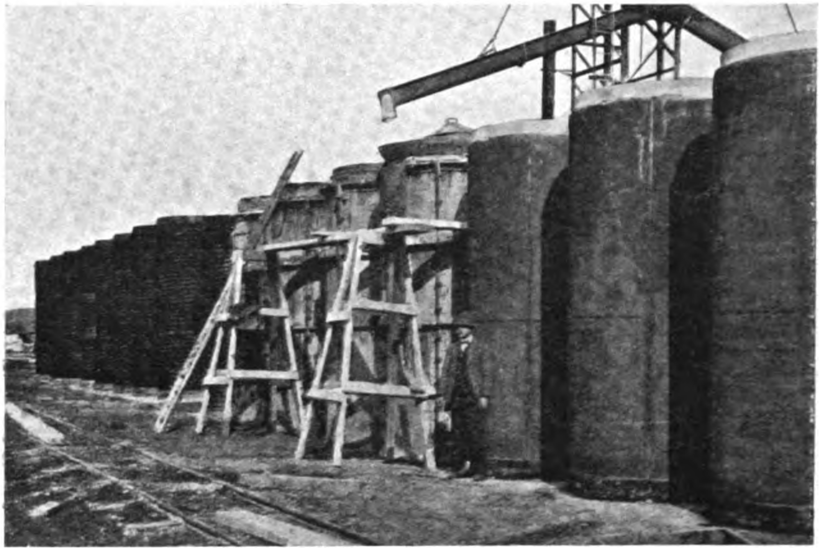
The two balconies of the theatre are each supported by main box-type heavy plated girders spanning from side to side of the structure. No supporting columns are visible, and the auditorium has an unobstructed space for vision. The steel rakers supporting the flooring to the balconies are R.S. joists running radially towards the centre of the auditorium, which gives a satisfactory and efficient seating arrangement. At their inner end the rakers terminate in cantilever frames composed of angle-iron sections riveted to the main balcony girders. The free ends of these cantilevers are connected with R.S. channel section bent to circle, to which are fixed steel upright posts and rails to support the balustrading. The whole of the girders and frames were as far as practicable riveted at the works before despatch. The remaining joints were riveted up on the site with pneumatic power hammers—a special air-compressor plant being installed on the job for this purpose. Altogether the total weight of steelwork erected amounts to approximately 700 tons.

The Architect is Mr. Percy Hothersall.

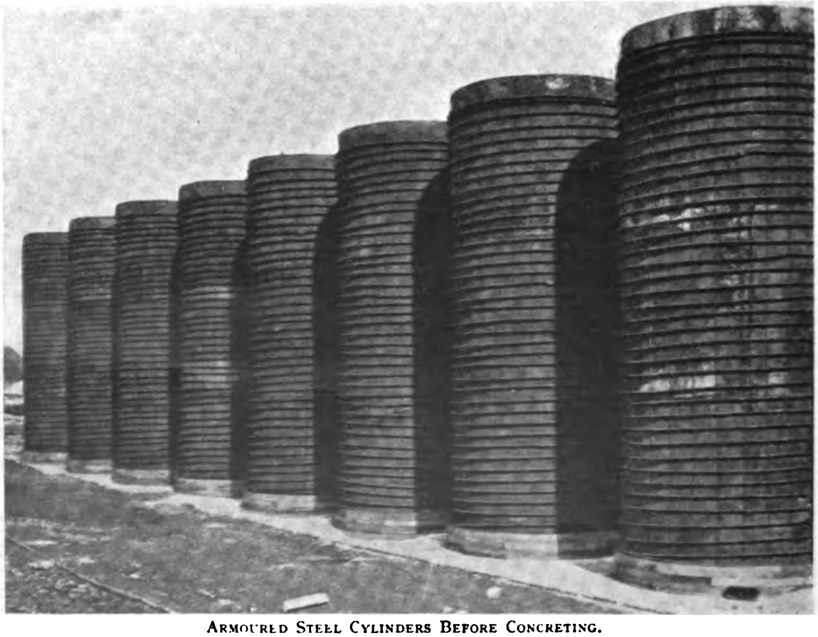
Messrs. Lambourne & Co., Ltd., constructional engineers, of Manchester and London, designed the whole of the steelwork and carried out the manufacture at their Openshaw workshops, and finally erected the steel skeleton framework on the site.



PICCADILLY PICTURE THEATRE, MANCHESTER: BALCONIES UNDER CONSTRUCTION.



Left: Armoured Tubes; **Centre:** Tubes Ready for Pouring Concrete; **Right:** Finished Tubes.



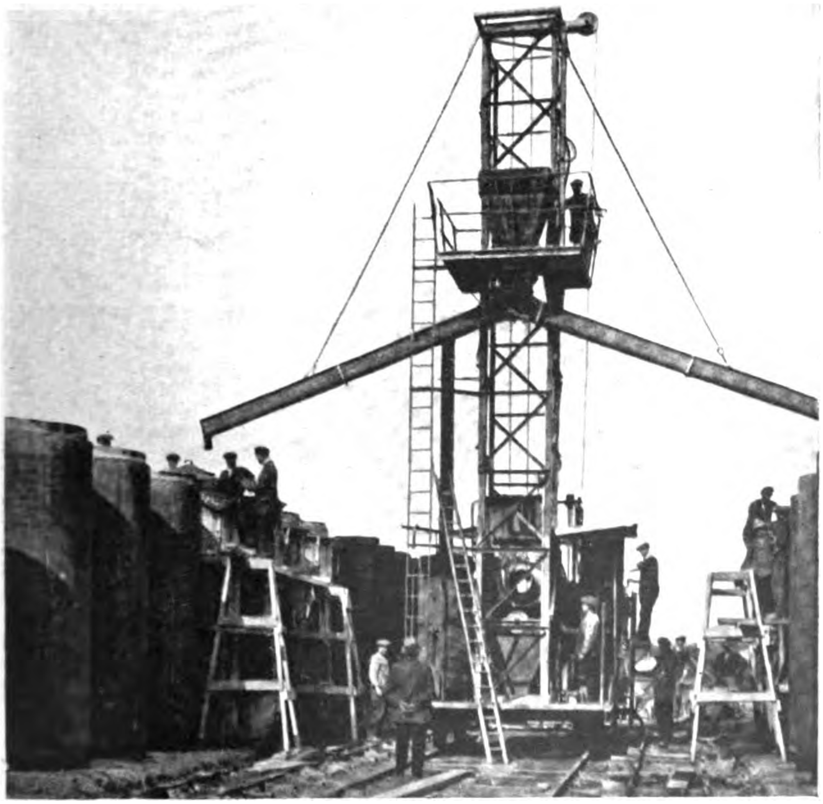
Armoured Steel Cylinders Before Concreting.

CONCRETE TUBES ON THE MANCHESTER NEW WATER SUPPLY SCHEME. (*See p.* 97.)

CONCRETE PIPES ON MANCHESTER NEW WATER SUPPLY SCHEME.

THE scheme of the Manchester Corporation for laying another pipe-line from Thirlmere, in order to augment the city's water supply for the future, is now in active operation. Thirlmere, or Leatheswater, reservoir has a capacity of over 8,000 million gallons of water, and at present supplies 50 million gallons a day, but steps have been taken to provide such a margin that for some time both industrial and domestic requirements will be amply served. The present undertaking is expected to be completed in about two years, the contracts for the different sections on the pipe track having been let to various firms. Thirlmere, which lies at the foot of Helvellyn on the north-west side, has been the chief source of the Manchester water supply for over twenty-five years. In 1890–94 a dam 800 ft. long was constructed in the valley, which was thus closed, and aqueducts were built to convey the water ninety-five miles to Manchester.

The scheme includes the laying of a reinforced concrete pipe between the valve-house on the Preston–Longridge road and Barnsfold Reservoir, close to Beacon Fell, a distance of approximately four miles. Work on this section at the north end of the Ribble siphon was commenced in November last on a site covering about six acres, and adjoining the London and North-Western railway from Preston to Longridge, a little over a mile up the line from Grimsargh Station

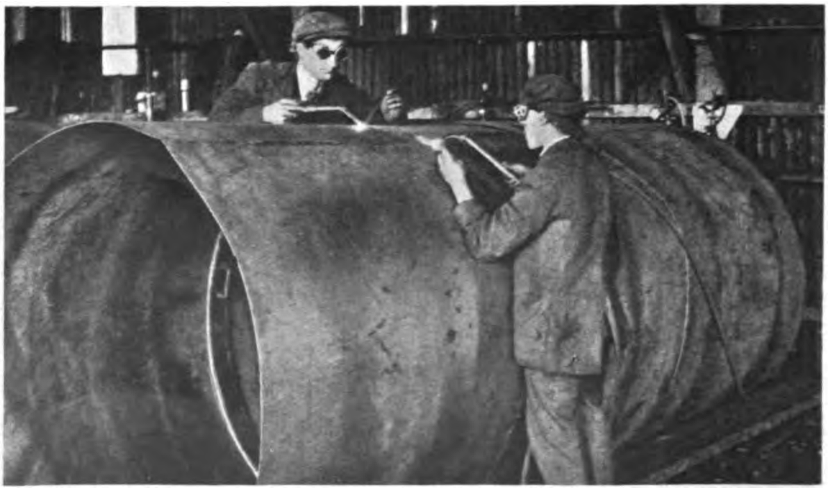


MIXING AND POURING PLANT.

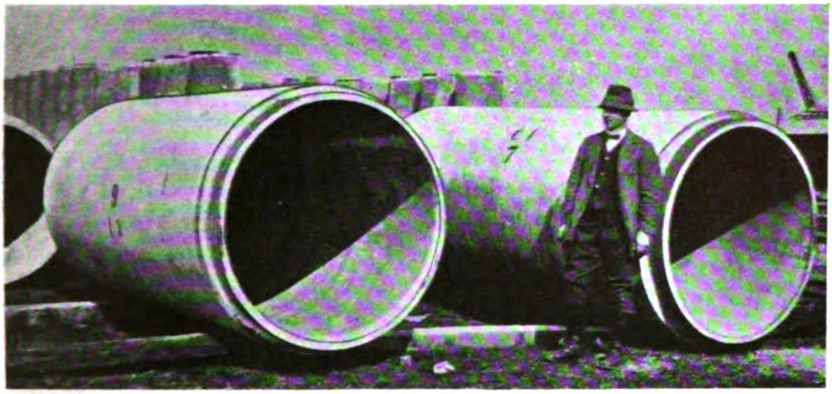
The contract for the manufacture of the concrete pipes and the laying of the main in the Longridge section was let to Messrs. Hughes and Lancaster, Ltd., Westminster, represented on the site by Mr. Clayton S. Sparrow, and the work is being carried out under the direction of the Manchester Corporation Waterworks Engineer, Mr. L. Holmes-Lewis, M.Inst. C.E., represented on the site by Mr. W. Patteson, M.Inst.C.E.

The concrete pipes, which are manufactured on the spot, are made on the "Bonna" system. Apart from special classes of pipe, such as the double-bend, the junction, and branch pipes, four standard types are made, and about 2,000 are required to lay this section. In the first place, the pipe is built up round a thin tube of steel, the standard thickness of which is $\frac{1}{16}$ in. and capable of bearing a pressure of at least 60 lb. to the square inch. The standard pipe used on this work is 12 ft. in length with an internal diameter of 54 in., and is composed of three sheets, or sections, of steel, each of which after being accurately sheared to the correct size is rolled to the desired diameter. The edges are then butted together, and welded by the oxy-acetylene process. The three sections of tube are then welded together, thus forming one pipe 12 ft. long, and extra stiffness is obtained by affixing seams over the joints and steel collars inside the ends. The armouring of the pipe is next proceeded with, and a wooden drum, 12 ft. long, provided with end pieces and trunnions so that it can be revolved, is placed inside the tube. Horizontal steel bars are laid at regular intervals along the outside of the tube and held in position until the winding over them of a cruciform spiral reinforcement. At each intersection of the armouring and the bars the two are bound with wire, so that a fairly rigid cage of steel is formed, and acetylene welding fixes it to the sheet pipe. The inner armouring is constructed in much the same way, but it is made of lighter steel, since its duty is only to support the concrete lining. This is made separately, the steel lengths being wound spirally over bars fixed horizontally to a wooden drum similar to the other. This second cage, when drawn off its support, fits inside the steel tube, to which it is afterwards welded.

The casting of the pipe necessitates the use of an outside mould and of an internal collapsible steel core. They are cast in two rows, between which is laid a light railway for the passage of a mechanical concrete mixer. When casting is being done, the armoured steel cylinder is arranged vertical on a previously prepared bed, and after the inside core is inserted by the aid of a crane the external mould, usually in several pieces, is bolted round the outside armouring. The con-



WELDING SECTIONS OF STEEL TUBE.



FINISHED PIPES.

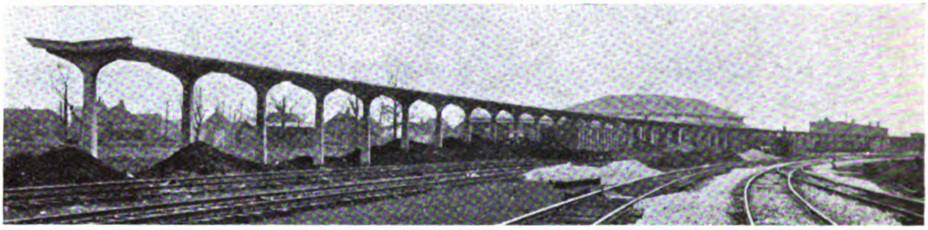
crete, composed of Penmaenmawr granite dust and cement, is poured into the mould from the two chutes of the mechanical mixer. The concrete, after being mixed, is placed in a hopper, which tips its contents into the distributing chutes on being hoisted to the required level. During the pouring process the concrete is consolidated by prodding it with rods, and the mould and core are removed as soon as the concrete has set, although the finished pipes are allowed to stand for two or three weeks until sufficiently hard to be laid and jointed.

In casting the concrete, the outer coating is stopped off at each end of the pipe for a distance of some 5½ in., the steel centre being thus exposed, and when the pipes are laid and butted end to end there is, therefore, a recess 11 in. wide between the outer coverings of both pipes, the depth being 2 in. The joint is made by a steel collar, ¼ in. thick, over which is placed a steel ring, while, as a last precaution, a collar of concrete is cast over the whole joint at the time the pipes are actually laid. The strength of the joint is shown by the fact that during a test a pressure of 544 lb. per square inch, or a head of 1,255 ft. of water, was reached, and even then the joint was not ruptured, the test being brought to an abrupt conclusion by the packing of the plunger of the hydraulic pump being blown out.

Eight pipes are made each day, and after use the moulds are oiled to prevent the concrete sticking.

WATER-PROOFING CONCRETE.

THE Swiss Waterworks' Association some time ago appointed a commission to investigate the various means which could be used for making concrete watertight. It first tried clay puddle, but eventually agreed that its use was accompanied by many difficulties and uncertainties; it is hoped, however, that tests now being made with mixtures of clay and sand may lead to better results. After investigating other waterproofing materials the commission reached the conclusion that the quality of the work rather than that of any added material is the chief factor. In other words, if the cement and aggregates are in proper proportions, if they are thoroughly mixed and the concrete well placed, there is no need for any waterproofing agents. Experiments made on concrete discs, 27 in. in diameter and 9 in. thick, which were placed in a vessel containing water under a constant pressure of 70 lb. per sq. in., showed that the blocks which were highly permeable at first rapidly lost their permeability, though even after three months they were still permeable. A curious feature of the results is that the drop in permeability is not regular, but increases for a few hours at various intervals, though the drop is, on the whole, very pronounced.—*Schweizer Bauzeitung.*



Concrete Canopy at Danville Station, Ill.

CONCRETE CANOPIES IN RAILWAY WORK.

The accompanying illustration shows the concrete construction of the canopies at the station of the Chicago & Eastern Illinois R.R. at Danville, Ill. Both pre-cast and *in situ* concrete were used in the construction. The pre-cast work was complicated by the fact that the canopies were built on a curve, so that each roof slab had to be slightly trapezoidal. Care had to be taken in casting these slabs, but neither the curve nor the grade of the platform caused any great difficulty of erection. As compared with the cost of the work that was poured in place, the greatest saving in the pre-cast unit was in its erection and on the column forms. There was little if any saving in the cost of slab forms. The *in situ* concrete involved some intricate form work.

The total length of the canopies is approximately 1,000 ft.; 450 ft. of this was unit construction, and the balance, consisting of the double sheds in front of the main building, was of a width too great to be handled economically in this manner and was accordingly erected in place. The single sheds, of pre-cast units, were composed of slabs each 4½ in. thick, 11 ft. wide, and 16 ft. long. Each of these was built in two separate units 5 ft. 6 in. wide to facilitate handling. As the roof is V shape and drained to the centre, the inner edges of the slabs were bevelled to fit, while the outer edges were cast with a curved finish. These slabs rested on T-shape columns spaced 16 ft. on centres, and were anchored by four 1-in. dowels at each end; a 3-in. stirrup was also anchored at each end to assist in the erection. Wooden forms were used, and to obtain a smooth finish these were carefully finished and covered with a heavy building paper and brushed with paraffin oil before pouring the concrete. This method left a smooth and clean finish when the forms were removed. The holes for the dowels were formed by 1½-in. galvanised iron tubes. The reinforcing for the slabs consisted of triangular mesh under ½-in. rods. A ¾-in. layer of concrete was first placed on the forms, and the mesh and steel rods were then placed over this layer, thus holding the reinforcing in place without the use of chairs.

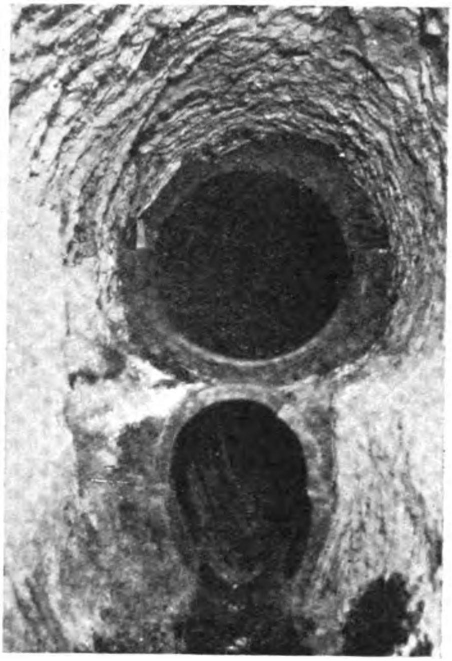
The T-shape columns are 12 ft. 8 in. high, having a 9 in. by 18 in. base, and a total breadth of 9 ft. 8 in. across the T. These rest on 9 in. by 18 in. piers, and four ½-in. rods extending 15 in. from the bottom of the columns formed dowels which were grouted in the slots in the piers. The reinforcing for the columns consists of six ½-in. square twisted longitudinal rods with ¾-in. hoops spaced from 4 in. to 12 in. on centres. Three ⅝-in. square twisted rods run across the top of the head and six ½-in. rods on each side run diagonally from these rods through the neck of the column. The columns were poured on carefully levelled platforms, the forms being placed so that the T is interlocked in order to save space. Eight sets of forms were used to pour the 32 columns. A single template was used in setting each column form so that all bearing edges for slabs would be the same on all columns. The curved part of the columns was formed by nailing sheet-iron on wooden frames cut to the proper shape. The greatest difficulty encountered here was in forming the bearing edge of the columns. Eight 1-in. dowels were set in this edge for engaging the slabs, and it was necessary to make this section in two parts so that it could be removed from the dowels without breaking the head of the column.

COMBINED SURFACE-WATER AND FOUL-WATER SEWER.

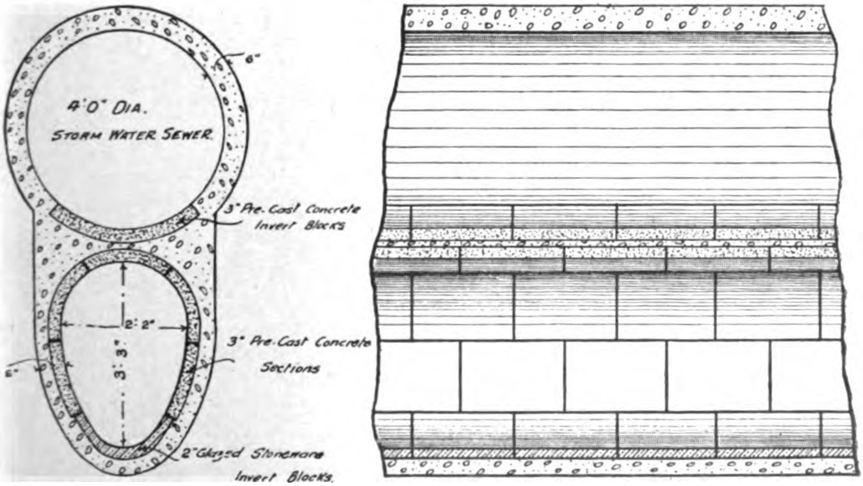
A NOVEL type of combined surface-water and foul-water sewer is being laid in Luton under the direction of the Borough Engineer, Mr. J. W. Tomlinson, A.M.I.C.E. In order to lay the pipe-line, which is to be two miles in length and to extend under the town at a depth varying from 19 ft. 2 in. to 48 ft. 2 in., manholes, lined with concrete, are sunk at intervals of from 200 ft. to 300 ft. and tunnels worked from both ends; in this connection it is interesting to note that so far the meeting places have not been more than ½ in. out of alignment.

The combination, as will be seen in the drawing, comprises a surface-water sewer 4 ft. in diameter and an egg-shaped foul-water sewer 3 ft. 3 in. in depth by 2 ft. 2 in. in width, and is laid in the following manner.

When the tunnelling between two manholes is complete a 2-in. glazed stoneware invert for the lower pipe is laid and bedded in 4 in. of concrete. The following day 3-in. pre-cast slab units are built up to form the pipe; ½-inch joints, which are broken, are made and the inside of the pipe carefully smoothed off. Care is exercised during



COMBINED SURFACE-WATER AND FOUL-WATER SEWER.


4'0" DIA.
STORM WATER SEWER
6"
3" Pre-Cast Concrete Invert Blocks
2' 2"
3' 3"
3" Pre-Cast Concrete Sections
6"
2" Glazed Stoneware Invert Blocks.

COMBINED SURFACE-WATER AND FOUL-WATER SEWER.

Manufacturing Pre-cast Slabs for Sewers.

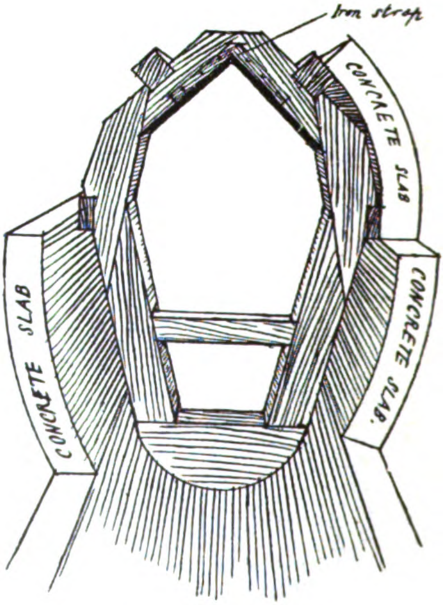
tunnelling to allow just about a 3-in. clearance from the outside of the slabs to the tunnel walls on both sides, and in these spaces concrete is filled and allowed to flow over the top slab and cover it by $\frac{3}{4}$ in. On this lies the 3-in. concrete invert, which is bedded in a similar manner to the stoneware invert. When the concrete forming the bedding has set the building of the surface-water pipe is commenced. Shuttering for the core, hinged at the bottom in order to make it collapsible, is placed in position. This core is in section a semi-circle of 4 ft. diameter, which allows half the pipe to be moulded in one operation; when the concrete for the bottom half has set the mould is drawn and replaced in an inverted position, thus forming the core for the top half. The walls of the pipe are 6 in. thick.

With the exception of the stoneware invert the whole of the work is of concrete, either in the form of pre-cast members or monolithic construction. The reason for the employment of this invert is that the glazed ware surface is known to withstand indefinitely without deterioration the constant scour of fine grit which takes place along a sewer invert.

The pre-cast members are manufactured in the Council's concrete yard, the aggregate being red granite graded 50 per cent. $\frac{1}{2}$-in., 25 per cent $\frac{1}{4}$-in. and 25 per cent. sand mixed with Portland cement in the proportion of 5 to 1.

As it is essential that each unit shall be perfectly true, the moulds in which they are cast are lined with steel plates; and in order to provide a very smooth surface for the interior of the pipe they are moulded on metal. The outside face is left rough to form a good key for the surrounding layer of concrete.

The soil through which the pipe-line runs is hard white chalk, which forms a very strong background, and reinforcement has not, therefore, been considered necessary in the construction of the sewer.



Formwork of Foul-Water Sewer.

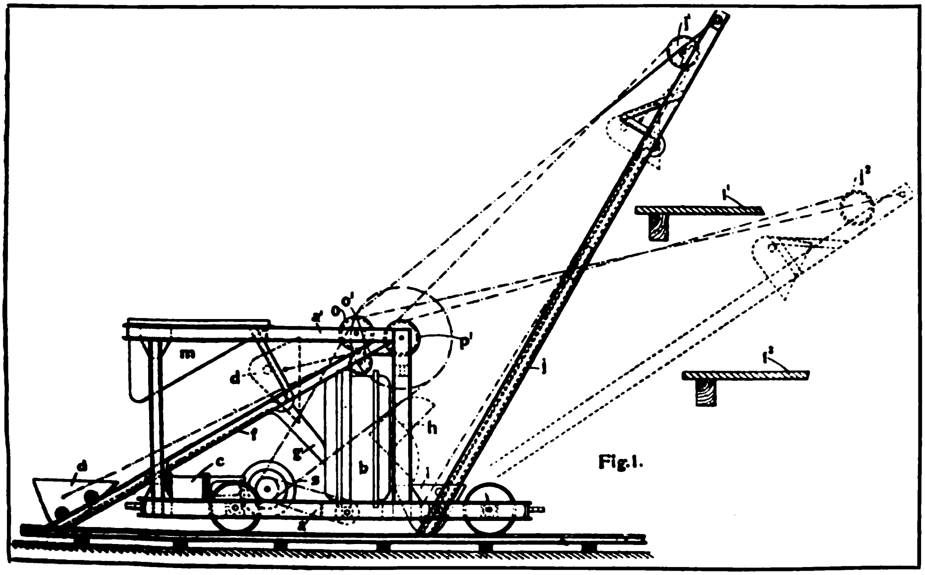
RECENT PATENT APPLICATIONS.

CONCRETE MIXERS.

[No. 183,548. J. W. Davison, M.I.M.E., Rhondda House, Pontypridd, and H. Bramwell, O.B.E., Pantyguesta, Llantrisant, Glamorganshire. April 21, 1921.]

This invention relates to concrete mixing machines more especially intended for arching in the underground roadways of collieries. According to the invention an apparatus is provided comprising an under-frame conveniently adapted to run on rails and having mounted upon it a rotary concrete mixer operated by a motor, the frame being a superstructure comprising rails upon which a charging skip may be drawn up from a low position convenient for being discharged from a colliery tram into a position in which its contents may be discharged into the mixer.

A steel frame (*a*), mounted on cast-iron track wheels and steel axles, supports a revolving mixing drum (*b*) of ordinary type driven by means of a counter-shaft with chains and bevel gearing from the motor (*c*). The mixing drum (*b*) may be disposed at a position near one end of the frame (*a*), while the motor (*c*) may be mounted near the opposite end. The detachable charging skip (*d*) is carried on a super-structure (a^1), and this skip (*d*) may be raised by means of a galvanised flexible wire rope from a winch (p^1), the rope passing over a loosely-mounted pulley or drum (o^1). A switch of known form is provided by which the skip (*d*) is caused to tilt as it moves upwards to the upper limit of its travel on the guides or rails (*f*) of the framing. By such means the concrete ingredients are charged into the drum (*b*) by means of a feed spout (*g*) mounted to the rear of the mixing drum. In front of the mixing drum (*b*) a swinging spout (*h*) is pivotally mounted and capable by means of a lever of being brought into an inclined position within the drum (*b*) so that in the rotation of the drum (*b*) the concrete may fall upon the swinging spout (*h*) and thus slide into the unloading skip (*i*) at the lowermost position of its travel on the jib (*j*). On the winch spindle two friction clutches may be mounted one for each drum, these being so arranged that either drum may be operated as required, the drums being free on the spindle (*p*) and being capable of being held in any position by applying a band brake on the outside of the drums. In a modified construction a drum is provided for the purpose of lifting and lowering the jib and means are provided, such as a worm and a detachable ratchet-handle, for the purpose of effecting the rotation of the drum.



No. 183,548.—Concrete Mixer.

FRAMEWORK REINFORCEMENTS.

[No. 184,050.—H. J. Nowlan, 124, Stubbington Avenue, Portsmouth. July 27, 1921.]

In the improved reinforcement constructed in accordance with this invention successive sinuous shear reinforcements run between the top and bottom reinforcements in parallel planes. Each shear reinforcement is preferably connected at or near each apex to the longitudinal and transverse reinforcing bars or the like by a coil or clip which ties together the longitudinal bar, transverse bar, and shear member so as to allow them to turn relatively to enable the whole framework to fold flat for transport or to adjust the distance apart of the top and bottom reinforcements. Alternatively, the coil or clip may be constructed of such size that when it embraces the longitudinal and transverse bars and shear members they are held tightly together and form a stiff framework. Longitudinal reinforcing members (B and C) and transverse reinforcing members (D and E) are provided at top and bottom, and zig-zag members (F) run between top and bottom in parallel planes. In order to give rigidity to the framework, supplementary shear reinforcements may be provided running in directions transverse to the direction of the main shear reinforcements; these may take the form of cross-bracing members (G) provided at intervals, and may be of hoop iron or steel and provided with slots for engagement over the main reinforcing bars (B). Also diagonal bracing wires or the like (H) in a horizontal plane may be provided to give rigidity to the framework in panels of convenient size.

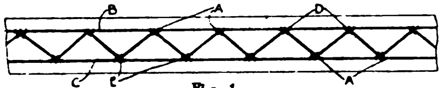


Fig. 1.

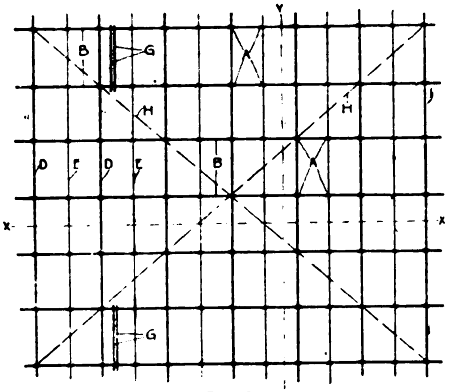


Fig. 2.

No. 184,050.—Framework Reinforcement.

CONCRETE WALLS.

[No. 184,273.—A. H. Barnes, 14, Sydenham Road, Croydon. May 10, 1921.]

The object of this invention is to provide a method of casting concrete walls which combines the advantage of the *in situ* method as regards avoiding a second handling with the provision of planes of weakness for concentrating expansion and shrinkage which occurs with the block wall. In building a wall in accordance with this method a course is first constructed of ordinary rectangular blocks (A) leaving between them gaps the length of which is about one-half the length of the blocks (A); the blocks (A) may either be pre-cast or cast ***in situ***. A series of blocks (B) of **T** form is then cast ***in situ***; these blocks have a total depth of two courses, and the length of each flange (B) may be about twice the width of the leg. The casting of the wall then

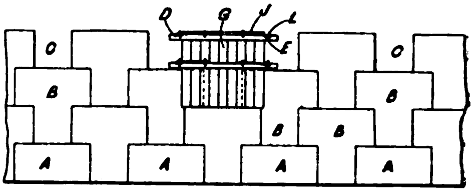


Fig1

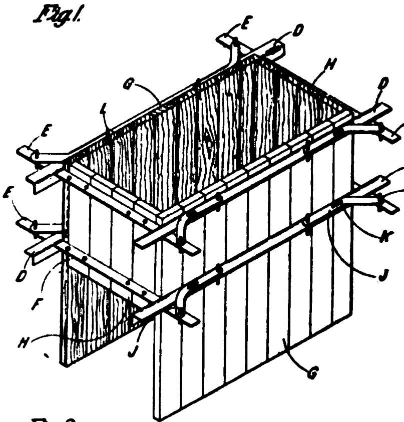


Fig2

No. 184,273.—Concrete Walls.

continues until the top course is reached, whereupon the gaps (C) in such course are filled in. A convenient form of mould for the blocks is built up of four longitudinal members (D) of angle steel and four transverse flat steel lateral members (E) passing through slots in the members (D). The lateral members are provided with spaced holes (F) to provide adjustment for varying thicknesses of walls, and the longitudinal members (D) are provided with catches (J) for engagement in the holes and tightening the side and end members of the mould together. These catches are pivoted at points (K) to the members (D), and are held in tightened position by pins (L). On releasing the pins the mould expands free from the work and can be moved without being taken apart. The members (D) carry boards or sheets (G) forming side members of the mould and two end members (H), of half depth, are provided and held in position by contact with the later members. The lower portions of the boards or sheets (G) bear against the projecting flanges of the T-blocks of the lower flange, and these facilitate vertical alignment of the wall. The invention is applicable to the formation of cavity walls by the use of cores in the mould, wall ties being inserted as the work proceeds.

BLOCK MOULDING MACHINES.

[No. 181,868.—W. R. Blackshaw, Denewood, Waterloo Place, Leamington Spa, Warwick. April 2, 1921.]

This invention comprises an improved machine of the kind in which a ram-plate is arranged to move into and out of a mould-box and in which the mould-box is movable by rollers running on rails under and away from the ram plate.

On the frame (1) of the machine is mounted a pair of guide rails (2, 2), for example, of inverted D-section. The bottom (12) of the mould-box (3) is provided on the underside with a pair of parallel recesses (4) into which the guide rails (2) enter when the mould-box is in position in the machine. The back of the mould-box and the bottom (12) are bolted together; the side members (14, 14) are hinged to the back at their rear edges and the front is hinged at its bottom edge to the bottom (12) of the box. The hinged parts are held in position by locking arms. Mounted on the mould-box at the rear are two rollers (6, 6) which are so disposed that they may be caused to engage the rails and support the mould-box when it is withdrawn from under the ram plate. These rollers are carried on a shaft supported on brackets at the rear of the mould-box. Mounted on the front of the mould-box

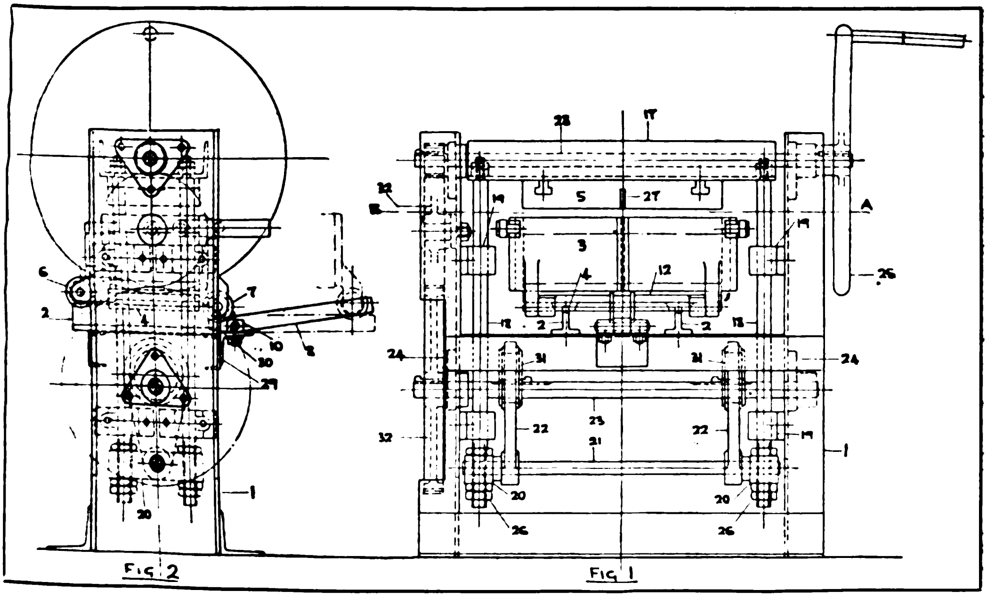


Fig 2

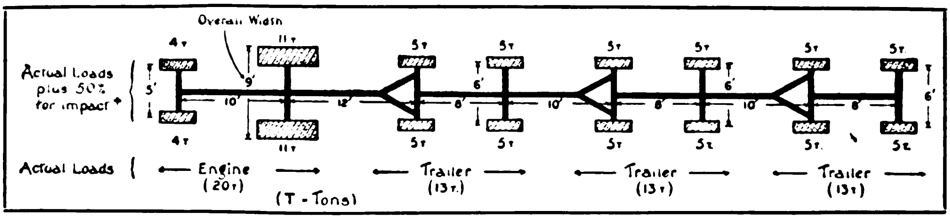
Fig 1

No. 181,868.—Block-moulding Machine.

at the middle is a third roller (7), which is so arranged that it is lower than the rollers (6, 6) when the mould-box is in position under the ram. On a pin mounted in bearings (10) carried on a bracket (29) is pivoted an arm or lever (8). The arm or lever is of such length that its short end is under the roller (7) at the front of the mould-box and at this end it is sloped as indicated at (30). The bracket (29) constitutes means to maintain the lever (8) in a horizontal or substantially horizontal position when the roller (7) passes beyond the pivot pin. The ram-plate (5) is supported from a pressure-plate (17), which is carried at its corners on vertical rods (18) mounted in bearings or guides (19) carried on the main frame (1). To the lower ends of each pair of rods is secured a bracket (20) by which is carried a spindle (21), on which are mounted, one at each end, rods (22) each having an eccentric (31) mounted on a horizontal shaft (23). The shaft (23) is supported in bearings (24) on the main frame (1) and is driven through reducing gearing (32) from a shaft (28) operated by the hand-wheel (25). Nuts (26) enable the movement of the ram-plate (5) into the mould-box to be adjusted.

STANDARD LOAD FOR HIGHWAY BRIDGES.

In its "Report on the Road Fund for the Year 1921-22" (London: H.M. Stationery Office, price 2*s.* 6*d.* net), the Ministry of Transport states that a large number of schemes have been under consideration during the year for the improvement, strengthening, or reconstruction of highway bridges, many of which fall considerably short of modern traffic requirements. "It is obviously desirable," says the report, "for a uniform standard of strength to be applied to bridges on all the principal highways, and with this end in view a standardised loading for highway bridges has been prepared (see diagram). The aim has been to ensure that any bridge, towards the cost of which a grant is made or a loan recommended, shall be so designed as to carry any traffic which may come on the road, so that no restriction need be imposed on the use of the bridge. In deciding upon this standard loading, it had to be borne in mind that a bridge is the most important link in the highway chain, and that a life of at least one hundred years should be anticipated. It was therefore essential that a margin should be allowed to cover the more onerous loading which may be expected in coming years. This standard loading is only applicable to bridges required to carry normal traffic, and in certain industrial districts, such as Liverpool, where heavy indivisible loads in excess of 16 tons have to be dealt with, the bridges must be designed of greater strength accordingly. These exceptional cases fall to be considered, in the first instance, by the highway authorities concerned. The load specified, namely, one traction engine and three trailers, is the heaviest dead load that is permitted for a train of vehicles having steel tyres, and to this dead load 50 per cent. has been added for impact. It must be admitted that 50 per cent. can only be regarded as an empirical figure, for which it is hoped to substitute an appropriate formula as the result of experiments which are now proceeding. The principal difficulty encountered by highway authorities in the matter of bridges arises in the case of those structures which are maintainable by undertakings such as canal companies, railway companies, etc., whose liability



Standard Load for Highway Bridges.

is limited to the maintenance of a structure adapted to the loads and traffic conditions of the period in which the bridge was erected. In these days it is evident that the greater number of canal bridges and many railway bridges need entire reconstruction. Not only is their strength inadequate, but in many cases the width is insufficient and the camber of the bridge unsuitable for fast-moving traffic. Continuous pressure is being exercised by the Department and many conferences have been held with a view to drawing up schemes for the apportionment of the cost of reconstruction between the undertaking, the highway authority and the Road Fund. In view of the magnitude of the problem some years must necessarily elapse before the highways of the country, especially in regions traversed by canals, can in this respect be brought up to a standard likely to give complete satisfaction to road users. In connection with the standardisation of bridge design, with regard to carriageways generally, a standard width is now recommended, based on a unit of 10 ft. per line of traffic. This dimension applies equally to the carriageway of highways and of bridges, and its adoption has been generally welcomed by local authorities. The introduction of such a standard will serve a useful purpose in discouraging the creation of carriageways of unnecessary width, as it is clearly undesirable that intermediate dimensions should be adopted which give an excessive width for (say) two streams of traffic, and an inadequate width for three."

In the diagram, the bridge is assumed to be carrying such standard loads or parts of standard loads as will produce the maximum stress in any bridge member or material under consideration, provided that in any train of loads there shall not be more than one engine per 70 ft. of the span of the bridge, and that the distance between the centre lines of two adjacent trains of loads shall be taken as 10 ft. in normal cases where the width of the carriageway is a multiple of 10 ft. Where a smaller multiple is adopted (but not less in any case than 8 ft.) it shall be assumed that the distance between the centre lines of two adjacent trains of loads is reduced to the dimension of the lesser multiple so adopted. The overall width of 9 ft. shown on the diagram must then be assumed to be reduced so as to leave 1 ft. clearance between the wheels of two adjacent trains.

ALEXANDRA SEA WALL, BRIDLINGTON.

[The Alexandra Sea Wall, Bridlington, illustrated above, must be one of the earliest examples of a reinforced concrete sea wall, as it was built in 1880; the engineers being Mr. R. St. George-Moore, M.Inst.C.E., and his late partner, Mr. C. G. Clarke. Unfortunately, the specification has been lost and the mixture of the concrete is uncertain. The reinforcement is shown on the drawings, however, and consists of horizontal layers of chain cables interlaced with vertical rods.]

CONCRETE BRIDGES WITHOUT CENTERING.

Monsieur G. Gouyaud has recently patented in France a method of building concrete arches without the use of centering, and claims thereby to effect a very great saving, as much of the wood and all the labour used in constructing the centering for one bridge is inapplicable to any succeeding ones. In the new method, the concrete arch is built in a tubular steel casing, which is then placed in position in the same manner as the principal cables of a suspension bridge, but with the advantage that the weight of the steel casing is much less than that of a cable for a bridge of the same span. The casing is supported by wire ropes attached to it and to one of two trussed pillars placed at each end of the bridge. These pillars and ropes are readily removed, and can be used repeatedly.

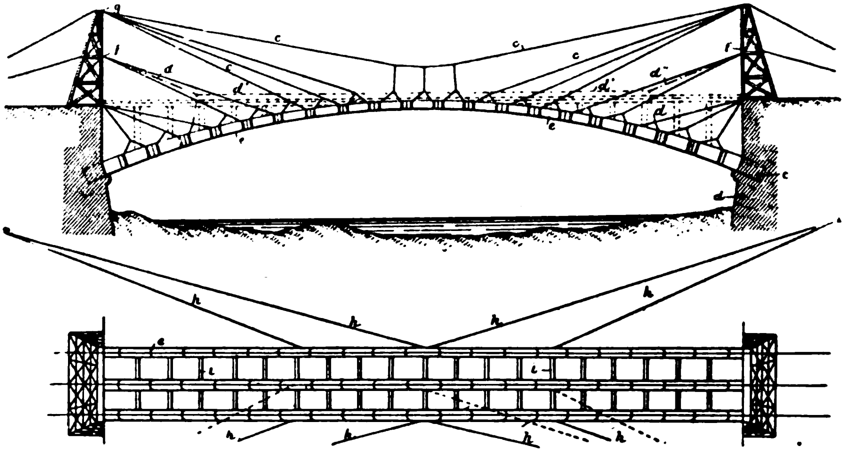


Fig. 1. Elevation and Plan of Steel Casing, Ropes, and Piers.

The steel casing is composed of a number of tubular segments (*Fig.* 2), their shape depending on the desired shape of the arch. The simplest shape is the plain cylindrical or oval tube, but two or more separate tubes may be tied together with struts or stays if a more complex arch is required. The segments are fastened together in such a manner as to permit the whole casing to be bent to the shape which it is desired the finished arch shall possess. When the steel casing is in place the reinforcement is put in its proper position in the casing, and the charges of concrete are

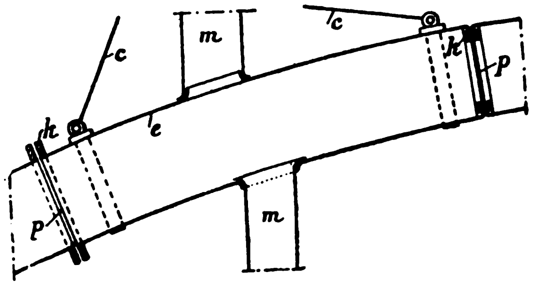


Fig. 2. Longitudinal Section of a Segment (*e*), showing Ropes (*c*), Flanges (*p*, *k*), and Ties (*m*).

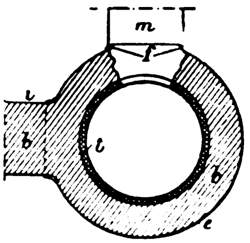


Fig. 3.—Transverse Section of Casing (*e*), showing Porous Pipe (*t*), Inlet for Concrete (*f*), Ties (*m*, *i*), and Concrete (*b*).

then added systematically in such a manner as to avoid overloading any one part of the casing. A convenient method consists of inserting a porous concrete pipe of such a size and shape as to fit the casing, and then pouring the concrete between this pipe and the steel casing. The concrete pipe is filled later. The concrete is consolidated by vibrating each segment mechanically. Any excess of water passes away through the porous pipe. When the casing is filled with concrete it may be covered with concrete to protect the metal, or if desired, it may be surrounded by a casing of reinforced concrete.

The plant needed for the erection of such an arch should include a powerful air compressor to deliver the concrete to the various parts of the bridge and to work the vibrating tools, etc. Electricity is also needed to solder the segments of the casing, to regulate the supporting cables, and generally to supply the power required. This method of construction is novel and it appears to have great possibilities, especially as it is capable of many adaptations.—*Le Genie Civil.*

DOUBLY-BURNED CEMENT.

Monsieur C. Didier claims to have secured a great improvement in the quality of cement by dividing the burning into two stages. The cement-mix is first heated to a temperature just sufficient to drive off all the carbon dioxide. The product is then hydrated, so as to produce an impalpable powder, and the latter is sieved and then burned as usual in a rotary kiln, or it is briquetted and burned in a shaft kiln. The process is considered to be specially advantageous where the cement is made from a hard limestone and a wet clay in a moist climate so that dry grinding of the raw materials is impracticable. In Monsieur Didier's process, the lightly calcined limestone or chalk does not need to be ground, as it falls to powder when mixed with the wet clay, simultaneously drying the latter. The saving effected in the power otherwise required for grinding is sufficient to make the process worth consideration, but in addition to this the process removes much useless material from the stone prior to the second burning and so produces a better cement.

A particularly interesting application of this process is to hydraulic limestones corresponding roughly in composition with the raw mix of Portland cement. If such materials are burned at a sufficiently high temperature they form natural cements, but these are usually inferior to Portland cement because few natural materials are as homogeneous as those which have been mixed artificially. The first burning is at too low a temperature to cause the complete formation of the desired compounds of lime and silica, but a large part of such combination does occur and during the slaking which follows these compounds are reduced to powder, whilst the inert impurities remain in a coarser form and can be separated by sieving.

This method of "double-burning" has been in use for more than a year at the Usines Valette-Viallard at Cruas, with complete satisfaction. The process separates inert and useless material at an early stage. Under some conditions this material is a kind of grappier, and may be used for "grappier cement." The quality of the cement produced in the second burning is superior to that produced by a single burning, because some of the impurities are removed and the materials are better mixed. The additional cost is not serious and is readily paid by those who realize the advantage of a superior cement.

When required the works can burn lime as well as produce cement, as the product of the first burning can, with a suitable stone, be a first-class lime. Incidentally, the sale of this lime in a hydrated state is very profitable. The conversion from the orthodox method to double-burning is quite simple and readily effected in all but the smallest cement works.—*Revue de Matériaux.*

LARGE PLANT IN CANAL CONSTRUCTION.

A FURTHER stage in the utilisation of the Niagara Falls for the generation of electricity was marked a few months ago, when the Chippewa–Queenston section was officially declared open. The construction of this section entailed a very large amount of concrete work, and the fullest use was made of labour-saving machinery. From the point of view of the civil engineer, the chief part of the work was the construction of a canal to take the water from the intake at Chippewa to the power house at Queenston, a distance of 12½ miles. With such a long canal, it was important that the retardation of the water by inequalities or roughness on the bottom and sides should be reduced to a minimum, and the whole of the canal has therefore been lined with concrete. This method of reducing the loss in the flow of water due to friction was adopted after careful consideration, and it is estimated that it has increased the capacity by over 20 per cent. A Canadian contemporary states that from the start it was obvious that something entirely unusual in the way of plant and constructional methods was necessary on account of the magnitude of the work, and in the determination of the type of plant and methods to be used the availability of cheap electric power, the fact that no less than 17,000,000 cu. yds. of earth and rock had to be removed, and the excellent facilities for the disposal of the spoil within a short distance of the work had to be taken into account. The excavation work was performed with fourteen power shovels, mostly electrically operated, five of which are claimed to be the largest ever built. This type weigh over 400 tons each, are operated by 750 h.p. motors, and have a capacity of 150,000 cu. yds. of earth or 70,000 cu. yds. of rock per month each. The largest shovel had an eight-cubic-yard bucket for earth and a six-cubic-yard bucket for rock. A car with a capacity of 20 cu. yds., standing 60 ft. above the shovel, was loaded, on an average, in 1½ minutes. The booms are about 90 ft. long, and the dippers 58 ft. The shovels are mounted on two tracks (four rails), on sixteen wheels. With the aid of such large shovels, supplemented with cableway excavators and other plant, some 3,710,000 cu. yds. of earth and 2,410,000 cu. yds. of rock were excavated in one year, that ending March, 1919, and it is claimed that a very considerable saving in cost has been effected as compared with the smaller units previously in use.

The concrete lining of the canal varies from 30 to 32 ft. in height. The floor covering, 6 in. thick, was laid first, and the lining of the sides then placed in position with the aid of special metal forms. This work was carried out by nine distinct " units," each comprising one concreting unit (steam mixer, hoist, spouts, and hoppers for stone, sand and cement), and two units carrying the shuttering. In operation, the lining was poured in 40-ft. sections, 6 in. thick, using the shuttering units alternately. On one section of the canal, the " whirlpool " section, which is trapezoidal in shape and the walls of which have a slope of 1 in 1½, a different procedure was followed, and the lining was heavily reinforced. Three-feet strips of concrete were first laid on the slope at 18-ft. centres, and the intervening spaces then filled in. Sliding metal forms were used, 16 ft. long by 17 ft. wide, consisting of one thickness of ¼-in. steel plate with a 9-in. channel around the outside, and reinforced with 9-in. channels placed horizontally across the plate at 9-in. centres.

GENNEVILLIERS POWER HOUSE.

The new electric power house at Gennevilliers, erected by the "Union d'Electricite," is stated to be the biggest works of its kind in Europe. It is laid out to house plant of 320,000 kilowatt capacity, and is designed to supersede a number of small generating stations. The site is on the banks of the Seine in a densely-populated district, close to the railway and situated conveniently for the delivery of fuel and stores.

Preliminary steps had to be taken to ascertain the composition of the sub-soil. As soon as the U.D.E. had acquired the land, in 1919, soundings were made, and it was found that, whilst the existing strata of gravel and sand would supply, at moderate depth, a very satisfactory basis for the foundations of the buildings as a whole, yet piling was necessary to carry the foundations of the heavy turbo-alternator sets and also the main piers of the boiler-house and of the engine-room building. The preparatory work consisted of :—(*a*) Sinking two wells on each side of the site of the engine-room, so as to drain completely the whole of the excavations of the site; (*b*) driving three timber test-piles with a view to determining the depth at which further penetration would be impossible, and to determine the length of the reinforced concrete piles.

All the experiments made seemed to confirm fully the results of the first examination, i.e., that the stratum of hard marl and clay on which the piles would come to a stop would be found in the vicinity of 18 metres. As a matter of fact the three piles driven in found a resistance of more than 40 tons at depths varying between 18·22 and 18·54 metres. The work of sinking simultaneously the two drainage wells met with very great difficulties, further intensified when it was found in the course of the work of excavation undertaken very shortly afterwards that the resistance to compression of the soil was much lower than originally supposed, and completely inadequate to support the weight of the heavy construction of the station. Recourse had, therefore, to be had to piling to a much greater extent than was originally intended, and the foundations of the main walls of the building had to be placed entirely on piles. It was of importance that the work of piling should be carried out with speed.

Fig. 1 shows the general appearance of

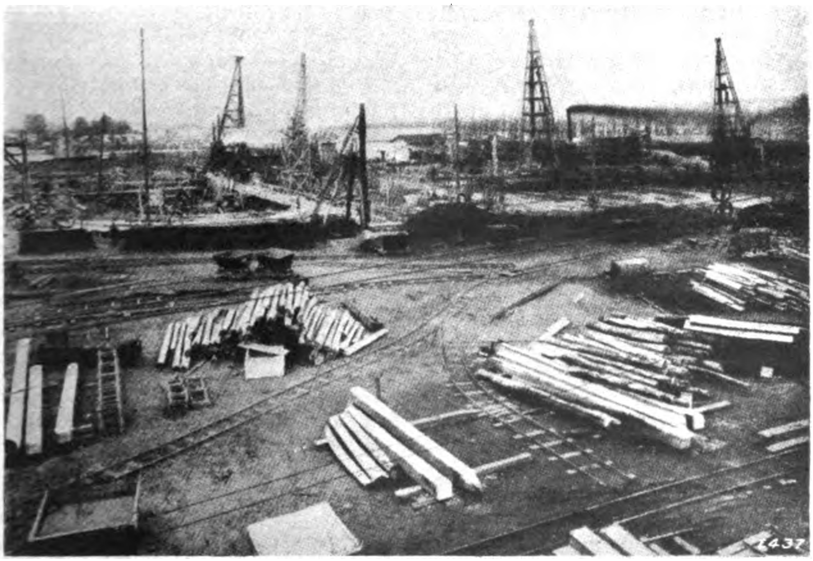


Fig. 1. General View of the Site in January, 1921, Showing Four Pile-drivers in Action.
GENNEVILLIERS POWER STATION.

the site during the days which followed the disappointing result of the first work of preparation of the site. A more thorough investigation showed that the hard layer, found at a depth of 18 metres, was too patchy and altogether too thin to be relied upon, and that the layers of sand constituted a much more serious obstacle than had been thought at first, as exemplified by the fact that the ramming of certain piles had to be stopped at a depth of 12 metres, whilst in other cases no support was found until a depth of 19 to 23 metres was reached.

The manufacture of the concrete piles, carried out on the spot, reached an output of about 10 piles of 6 to 7 metres length per day; this was increased subsequently to 30 piles per day, and the length of the piles had to be increased to 12 and 14 metres. The period of drying had also been reduced to a minimum, and the contractors were compelled in many cases to drive piles made only eight days previously under loads of 1½ tons, and under a height of fall which could not exceed 1 metre. The heads of the piles were smashed in a number of cases; and more delay was experienced when the heads of these piles had to be re-made with metallic cement and the pile allowed to dry again for 48 hours. The number of pile-drivers at work had to be increased to five. In spite of these set-backs, the manufacture of all the piles for the site of the main buildings was terminated on January 15, 1921. Altogether 2,499 piles were made, 639 of which were more than 14 metres long, 329 more than 12 metres long, and the rest 5 to 7 metres long. The last of the pile-drivers had done its work by January 6, 1921.

The time of delivery was very short, and the difficulties experienced compelled the contractors to devise ways and means for hastening the date of termination of the buildings as soon as the foundations would be sufficiently advanced. It was decided to erect the most indispensable buildings (boiler-house and engine-room) in vertical elevation, instead of following the usual plan of simultaneous horizontal construction of the central body of buildings. Once this organisation of the work had been decided upon, the progress made was of a satisfactory character, and may be summarised as follows, from the point of view of the concrete work only, at intervals of three months:—

On January 1, 1921:—Boiler-house: Piles driven in and concrete base-slabs of the walls run. Engine-room: Eastern, western and central walls, with their piers to the level of the ground. Water-gate: Concrete run into the excavations of the sluices.

On April 1, 1921:—Battering of piles: Concrete work of the piers of the boiler-house. Engine-room: Work on the bases of the metallic stanchions; piers raised to a height of from ground-level to 7 metres above ground-level. Accumulator room: Built up to a height of 2½ metres above ground-level. Water-gate: Concrete bottom of the sluice run.

On July 1, 1921:—Reinforced concrete work of the boiler-house. Boiler housings. False-work of reinforced concrete floors. Engine-room walls nearly finished. Water-duct finished on a length of 50 metres. Accumulator-room walls ready to a height of 2½ metres; front facing the river finished (0·50 metres) and concreted.

On October 1, 1921:—Concrete work of the boiler-house practically ready. Engine-room: Three sections out of four ready. Accumulator room: Floors and watertight finish of the first five rooms ready; transformer cubicles finished in the three first spaces of 11 metres. Concrete work of the frontage terminated. Low-tension building practically ready. Water-ducts from the sluices to the engine-room finished. Twelve metres of reinforced concrete of the quay-side. Preparatory work for dam in the Seine. (See *Figs.* 1 and 2).

On January 1, 1922:—Boiler-house, accumulator-room, low-tension switchgear house and water-ducts completely finished. Engine-room: Four sections out of five ready.

The work on the sluices of the water-supply was taken in hand on July 30, 1920. It was executed behind a screen of Ransome metallic sheet-piling, raised parallel with the river, at a distance of about 1 metre from the bank and provided with right-angle wings at both ends. Behind this sheet of piling the earth was excavated, the pit being kept dry by means of a special pumping station installed at the bottom of a well sunk for the purpose. The sluice-work finished in October, 1920.

AN AMERICAN ARCHED DAM WITH TANGENT ENDS.

THE arch type of masonry dam has met with much favour in America and has been used for a number of structures. According to report there is no record of any dam of this type having failed under water pressure. A novel and important development is the design and construction of a concrete dam having its central portion arched, but abutting against tangent extensions in the upper and longer part of the structure instead of against the rock walls of the gorge. This combination type has been employed in the East Canyon Creek Dam, in Utah, which stores water for a large irrigation system. The section of the dam is peculiar also in that it has the batter on the up-stream face in the lower portion and on the down-stream face in the upper portion.

This dam, with its unusual design, has been described by its designer, Mr. A. F. Parker, in the *Proceedings* of the American Society of Civil Engineers. The length of the dam on the crest is 273 ft., the central arched portion being 128 ft. long, the north tangent 102 ft. long and the south tangent 43 ft. in length. The maximum height is 190 ft., the base on bed rock being about 45 ft. below the bed of the stream, or 50 ft. below the assumed datum. For 40 ft. from the base the thickness is 28 ft. to 30 ft., as only the heavy timbers were removed from the excavation and the whole space between the lagging was filled with concrete. Forms began at this 40 ft. level, with a thickness of 26·25 ft.

To fit the arch to the cliffs in the lower portion of the dam it was necessary to adopt radii of 70 ft. to 98·8 ft., and at 60 ft. above datum to change the batter from the up-stream to the down-stream side, the latter side being vertical up to that level. A top thickness of 5 ft. was adopted and on the down-stream side of the top there is an 8 in. rib 4 ft. deep, which was put in mainly for appearance. The top is protected by a parapet or splash wall on the up-stream side and by a railing on the down-stream

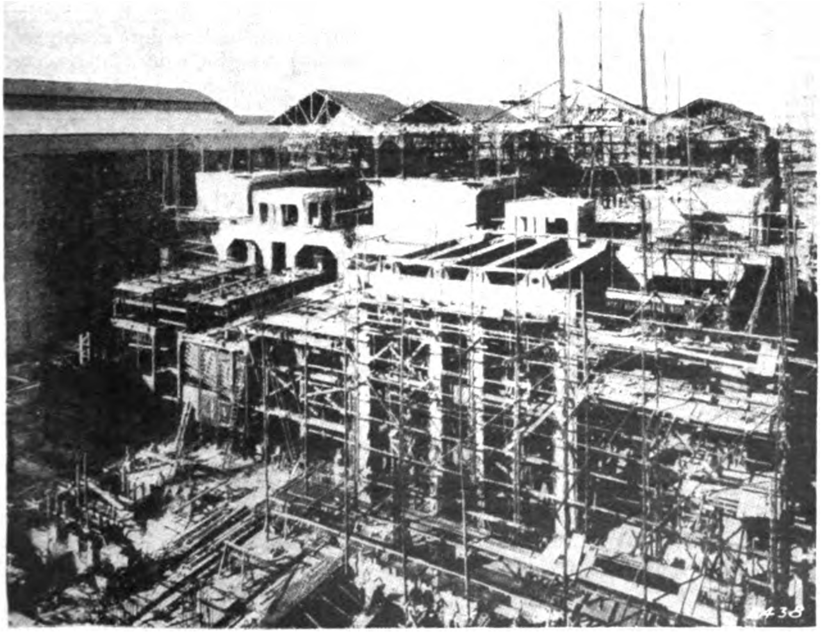


FIG. 2. GENERAL VIEW OF BOILER HOUSE UNDER CONSTRUCTION (OCTOBER, 1921). GENNEVILLIERS POWER STATION. (*See* p. 111.)

side. As the sides of the canyon are nearly parallel with the entire line of the inclined angle of the arch, while cutting into the rock to give arch abutments on radial lines would have been expensive, it was decided not to blast out more rock than was necessary to secure solid abutments. Steel dowels 4 ft. long, spaced 4 ft. horizontally and vertically, were grouted into 2 ft. holes in the rock face of these abutments, although Mr. Parker says: " Such dowels may not do much good, but they certainly give a look of security."

Site conditions indicated the desirability of the arch type and calculations showed that an arch dam could be built for about half the cost of any other type.

In the design various mathematical treatments were studied, and it was found that in all cases the dimensions necessary to resist the complicated shear, bending and temperature stresses did not exceed the dimensions computed from the simple formula for the strength of cylinders under pressure, when the unit stress on the masonry was conservative. In existing arch dams the stress was found to range from 10 to 61 tons per square foot, and for the East Canyon Creek Dam 12·5 tons was adopted. The thickness for the arch, in feet, was determined by the simple formula of multiplying the up-stream radius R in feet by the water pressure P in tons per square foot and dividing the product by the stress on the masonry. For practical use the thickness at any depth D for this stress $= 0{\cdot}0025\, R\, D$. The assumed compressive strength is 150 tons per square foot and a factor of safety of 12 is allowed. As to the factor of safety, Mr. Parker says that until some arch dam fails under pressure there is nothing to show the limit between bold and conservative design.

To avoid a long radius for the dam, which would have made the ends of the arch too nearly parallel with the sides of the canyon, it was necessary to employ tangent ends for the upper and longer portion of the dam. At the north end the rock fell away in such a manner that a hollow deck construction was necessary in order that the sloping side could be on the up-stream face. The deck is formed by arches in the lower part, with a straight surface above, set on piers spaced 7 ft. on centres. Similar construction was intended for the shorter south tangent, but after the foundation for it was blasted out the contour was changed so much that a gravity section was substituted, having a batter on both sides. Dowel pins were used in the abutments of the lower part of the dam.

To transmit the thrust of the arch dam through the hollow north tangent two heavy struts or ribs, extending from pier to pier and to the side of the canyon, were placed. The lower strut is 7 ft. wide and 4 ft. high, while the upper one is 6¼ ft. by 3 ft. At the top the thrust is taken by the floor of the spillway, which is formed in the north tangent. This spillway is 42 ft. wide and 5 ft. deep. It is fitted with swinging steel supports for plank flashboards. The supports are hinged at the top and spaced 3½ ft. on centres. The inclined apron extends from the crest to the upper one of the two struts mentioned above and is so formed as to throw the water clear of the dam.

Contraction joints are placed only where the arch abuts against the tangents. As these joints are on radial lines no dowels or rods extend across them, but heavy offsets were made in the concrete.

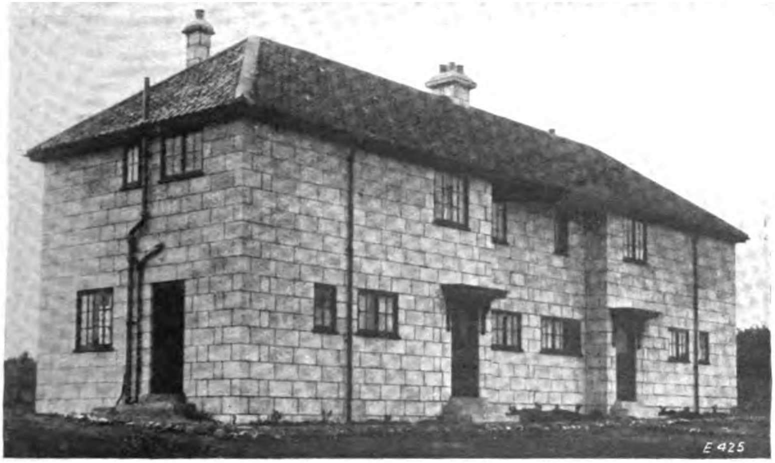
Steel reinforcement was not used until a certain elevation was reached, but from that point to the top bars of 1 in. twisted steel were placed 2 ft. apart in both directions and 1 ft. back from the face of the concrete on both sides of the arch section. Similar ½ in. and ¾ in. rods are used in the deck type of the north tangent. This arrangement of the steel was adopted as a compromise of apparent demands of the elements involved—the horizontal steel to stiffen the thinner parts of the arch, the vertical bars to distribute ice pressure and all the reinforcement serving to prevent cracks. Mr. Parker states that the reinforcement has certainly prevented cracks, but that the steel may be in greater quantity than is really required. On the other hand, the heavy reinforcement may be warranted as a measure of safety in view of the ice thrust against the dam, as ice of great thickness forms there during the severe winter weather.

CONCRETE COTTAGE BUILDING.

HOUSES AT GREAT MISSENDEN.

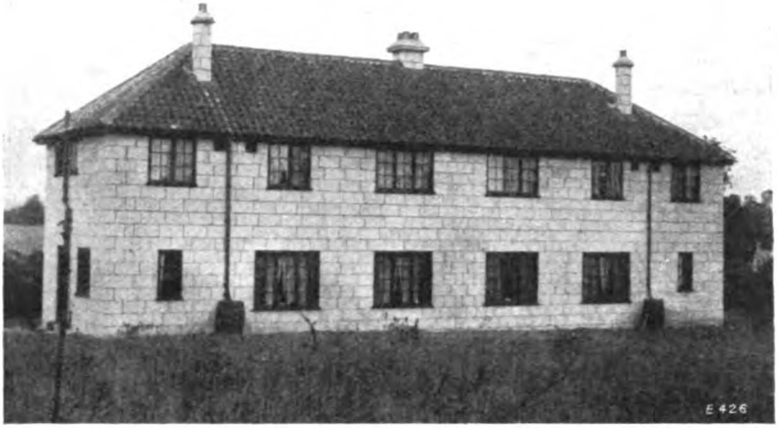
ALTHOUGH the activity in cottage building has abated, there are many examples of the cheapness of concrete as against brick construction. Some time ago tenders were obtained for the pair of houses which are herewith illustrated, in brickwork roughcasted, and with breeze internal partitions. Despite the fact that only solid 9 in. walls were specified, the lowest tender was £2,400 for the pair. This was too high for acceptance, and it was decided to invite tenders for concrete instead of brickwork. A set of half-inch working drawings, detailing the building to block units and showing the size of every block, was prepared by the architect (Mr. J. Harold Sayner, A.R.I.B.A.), and the estimate of Messrs. Bell's United Asbestos Co., Ltd., was accepted at £2,100. Although the preparation of these drawings entailed a considerable amount of extra work, from the point of view of building efficiency and economy the expenditure was justified, as it enabled all the blocks to be made at Messrs. Bell's works near Rickmansworth and sent by lorry to the site, a distance of 14 miles, as and when required. Furthermore, although the operatives had never handled the material before, the work was able to proceed with great rapidity, the whole building from start to completion and occupation being carried out between the dates May 16 and August 23.

An interesting point about the construction of these cottages is to be seen in the first floor, which is framed and carried on wrought corbels supporting beams into which the joists run: by this method block cutting was avoided. The floor is of 1-in. grooved-and-tongued boarding. All the woodwork is treated with two coats of Solignum instead of paint. The party and outside walls are built of " Hurcan " slabs with a cavity; the cement mortar was composed of three parts sand to one of cement. Externally the slabs were pointed with a weathered joint mixed 2 to 1, and internally with a struck joint in parian, this latter



[*Mr. J. H. Sayner, A.R.I.B.A., Architect.*

HOUSES AT GREAT MISSENDEN: NORTH ELEVATION.



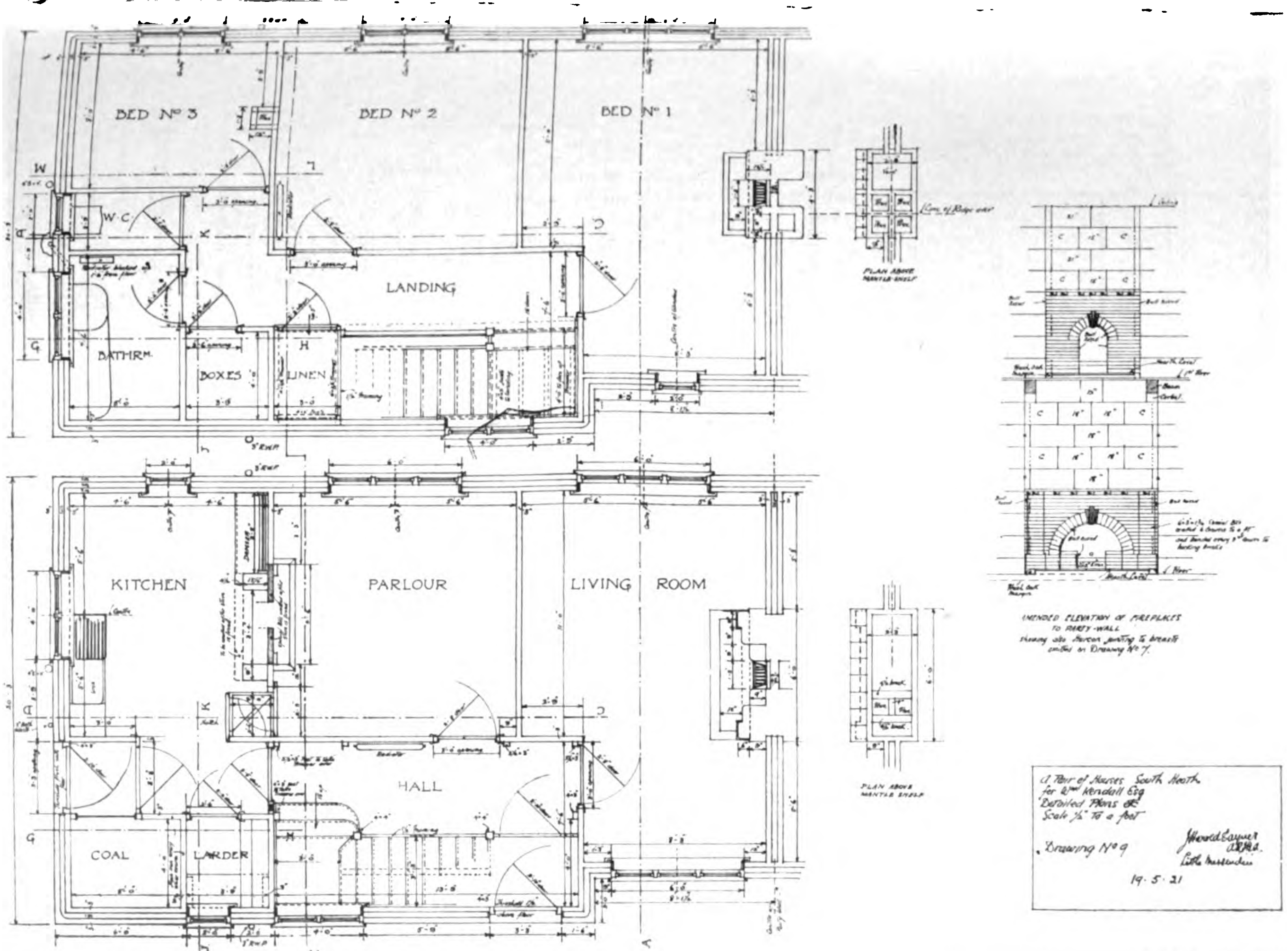
[*Mr. J. H. Sayner, A.R.I.B.A., Architect.*

HOUSES AT GREAT MISSENDEN: SOUTH ELEVATION.

operation being done after all the joinery was fixed. This elimination of internal plastering contributed not a little to the general saving. An item of expense, however, which does not occur with a brick-built house is worth noting. Owing to the necessity of avoiding putlog holes in the walls, independent scaffolding was used; this is an item that would be saved in a one-story building where a trestle is of sufficient height.

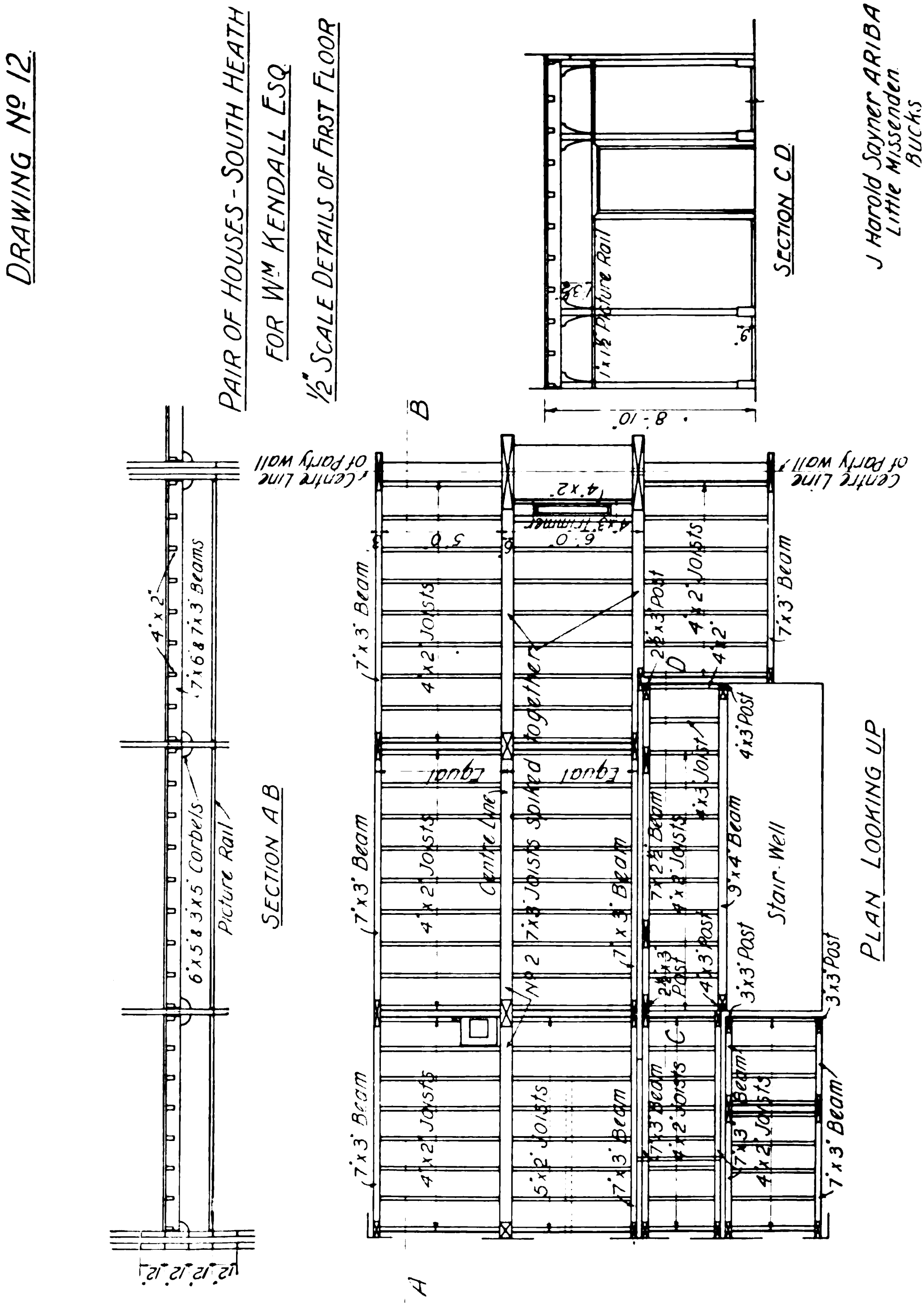
Decorative features have been made of the fireplaces in the sitting-room and large bedroom, which are constructed of special 6 by 2 multi-coloured sand-faced bricks with raised hearths and shelves of quarry tiles. Considerable care was bestowed on the question of heating, which, unfortunately, is often somewhat neglected, no steps being taken to improve on the cumbersome Yorkshire range and the open fire. Here, a combined central heating and cooking apparatus was fitted in each house between the kitchen and meal room, with an open fire to the latter—this supplies three radiators and the domestic hot water by contact with a copper coil in the feed tank, ensuring a constant supply to the sink, bath, and lavatory basin. This apparatus was supplied and fixed by the Hot-Water Stove Co. of Guildford.

The houses are roofed with selected old pantiles, whose warm colours contrast pleasantly with the cooler tones of the walls, a contrast which is repeated in the marked verticality of the pantiles against the horizontal courses of the blocks. On entering there is a small lobby with a cycle store on one side and coat recess on the other. There is a large sitting-room, and a smaller meal room arranged in conjunction with the kitchen, to which it is connected by a serving hatch. The kitchen is large and contains the sink and draining board, and is fitted with a cabinet—an improvement on the more usual open dresser. A back lobby gives access to the large fuel store. There is also a large larder. On the first floor are three bedrooms, all with south aspects, a large bathroom with bath and wash basin, a w.c., and, what will be particularly appreciated, a commodious linen cupboard and a box room. The contractors were Messrs. H. J. & A. Wright, of Great Missenden.



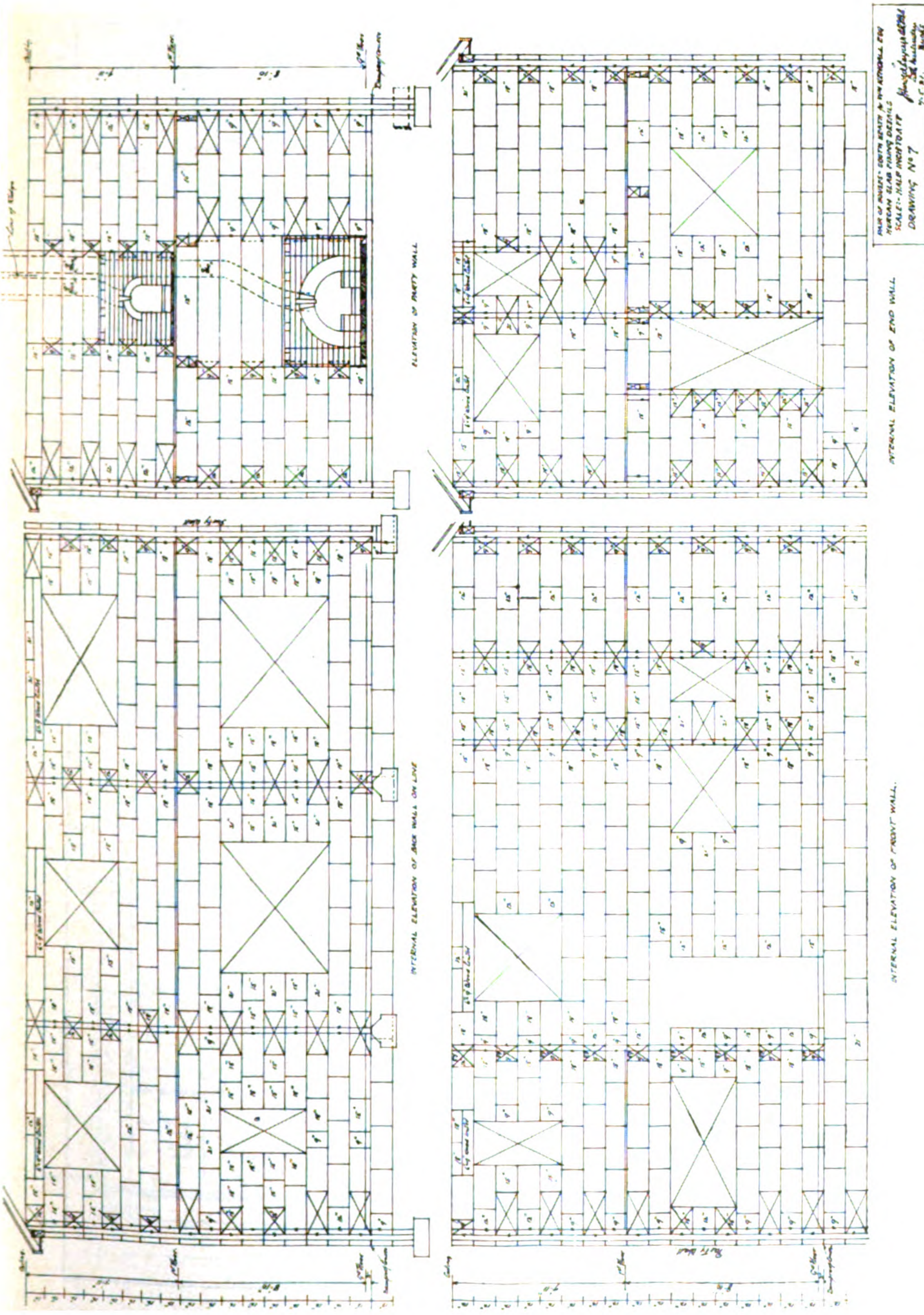
[Mr. J. Harold Sayner, A.R.I.B.A., Architect.

PAIR OF COTTAGES AT GREAT MISSENDEN: PLAN AND FIREPLACE DETAILS. (*See p.* 115.)



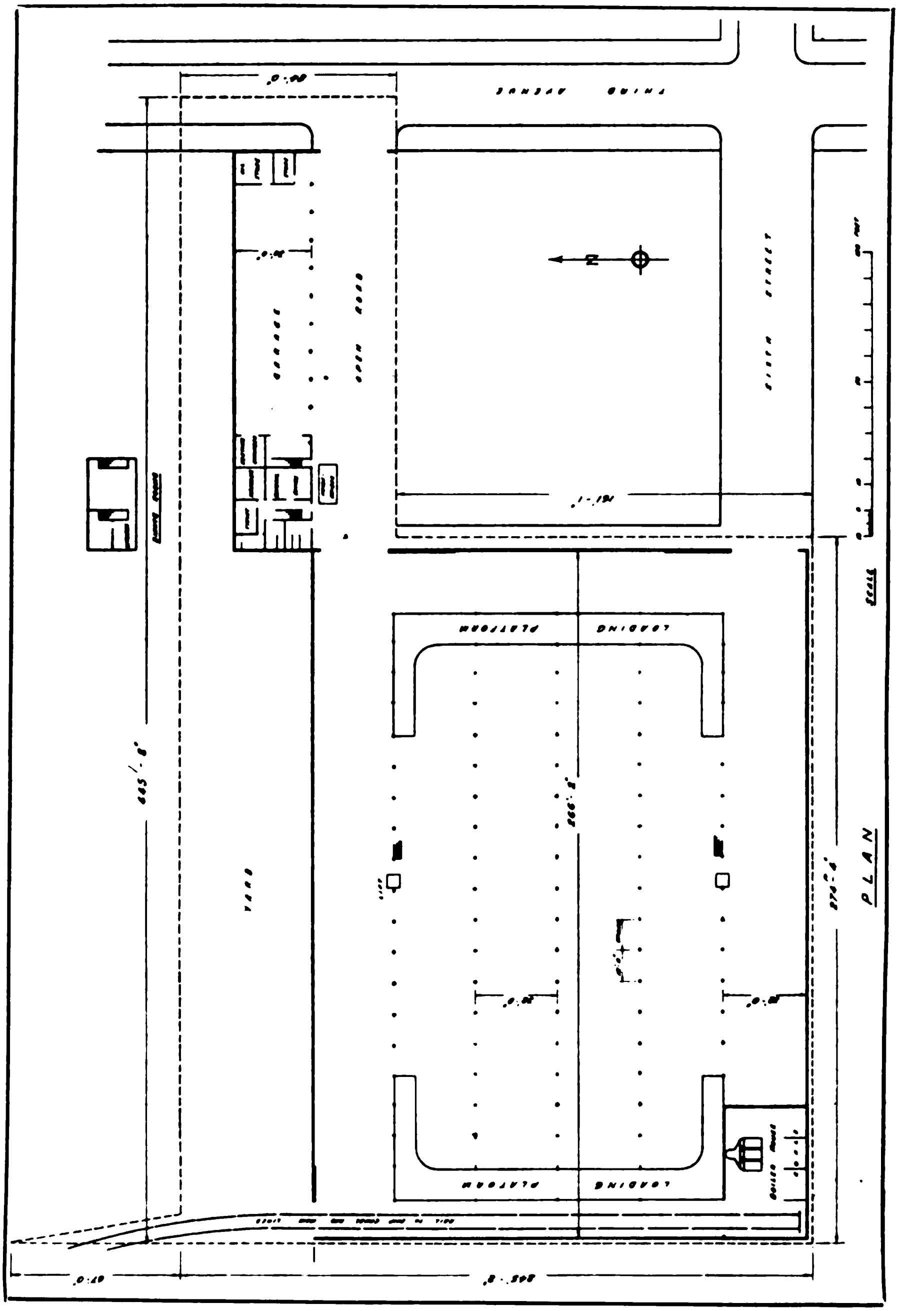
Pair of Cottages at Great Missenden: First-Floor Details. (See p. 115.)

Mr. J. Harold Sayner, A.R.I.B.A., Architect.


ELEVATION OF PARTY WALL
INTERNAL ELEVATION OF END WALL
INTERNAL ELEVATION OF BACK WALL ON LINE
INTERNAL ELEVATION OF FRONT WALL
PAIR OF HOUSES · SOUTH HEATH for W.M. KITCHENALL ESQ
HERCULEAN SLAB FIXING DETAILS
SCALE:- HALF INCH TO A FT
DRAWING No 7

PAIR OF COTTAGES AT GREAT MISSENDEN: BLOCK DETAILS. (See p. 115)

[Mr. Harold J. Sayner, A.R.I.B.A., Architect.



[Mr. Arthur Clayton, M.S.A., Architect.

NEW WAREHOUSE IN MANCHESTER. (See p. 121.)

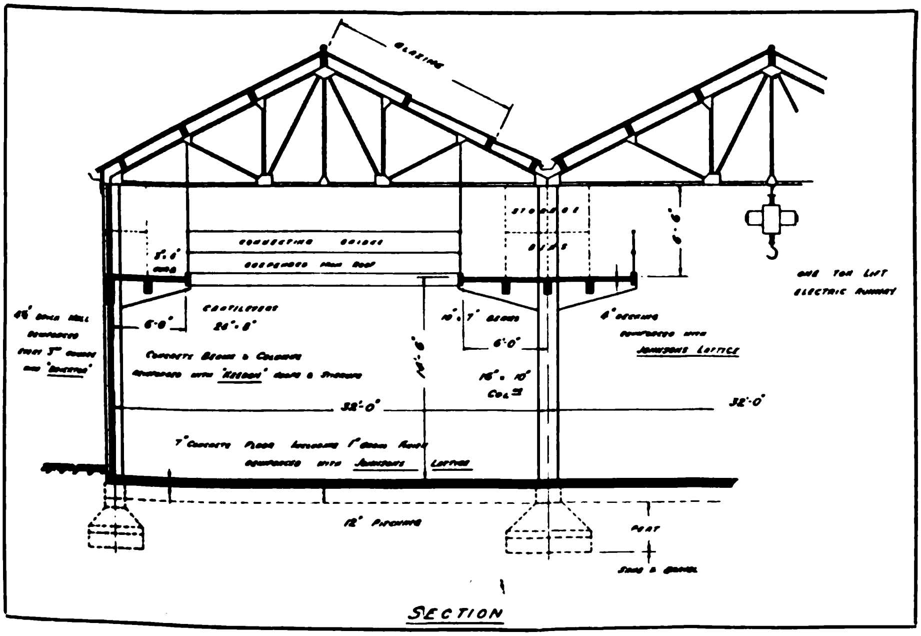
LARGE NEW MANCHESTER WAREHOUSE.

Owing to their increasing business, Messrs. Rd. Johnson Clapham & Morris, Ltd., of Manchester, have found it necessary from time to time to increase their warehouse accommodation, and as the Lever-street premises could not be extended, and, moreover, it being an expensive site without railway facilities, accommodation had to be sought elsewhere, which resulted in the renting of several warehouses isolated from one another. This, naturally, was unsatisfactory, and it was decided by the Directors to erect a new and commodious warehouse on modern lines, with garage and office conveniences of the latest ideas. A site was chosen at Trafford Park having good road and railway facilities, lighting, and drainage. The new warehouse will cover an area of 9,000 sq. yds., and is of the single-story type, 21 ft. high to the lowest point of the roof, with galleries 14 ft. 6 in. high.

The adoption of galleries in warehouse construction is believed to be unique, and gives greatly increased storage area under one roof; the galleries are to be used for the storage of smaller articles in bins, while the more bulky and heavy material will be kept on the floor. These galleries, of cantilever construction, are connected by bridges at intervals, and have hoists and staircases to facilitate the rapid selection of goods.

The building will be principally constructed of reinforced concrete, with light steel roof trusses covered with asbestos slates. There will be six bays each 32 ft. long, provision being made for electric runway cranes; in addition, there will be electric trolleys operating on the floor working from a ramp at the roadway and railway loading platforms. As motor lorries are extensively used for the delivery of orders, the loading way is arranged with the entrance on one side and exit the other, thus preventing congestion, while the railway is at the opposite end of the building. The building will be heated on the intermediate-pressure hot-water system, and provision made for adequate ventilation.

Drawings and schemes for the building were prepared by Messrs. Johnsons' Reinforced Concrete Engineering Co., Ltd. (a subsidiary company), in conjunction with Mr. Arthur Clayton, M.S.A., architect, of Manchester. The contract has been let to Messrs. James Byrom, Ltd., of Bury, for the sum of £30,000.



New Warehouse in Manchester.

CEMENT NOTES.

Soundness Tests For Portland Cement.

By the soundness of Portland cement is meant its freedom from expansion other than that caused by increase of temperature or by a change from moist to dry condition. Both of the latter are of such small proportions as to be negligible in constructional work, whereas a cement which is inherently unsound may have expansive properties sufficient to cause disintegration of the concrete containing it or to endanger the stability of a structure in which such concrete is embodied. For example, a concrete floor in which unsound cement is used will either force outward the walls between which it is placed or bulge upward until it cracks.

The damage caused by an unsound cement may be so great that tests for soundness have always been regarded as important, and much discussion has taken place as to suitable tests for the purpose. The most rational test is to prepare bars or cakes of the cement and to ascertain by measurement or observation over periods of months what expansion occurs either in air or water. Such procedure is, however, impracticable owing to the time taken, and consequently various " accelerated " tests have been devised. The basis of " accelerated " tests is the submission of the gauged cement to hot water, and the various methods have ranged from the Faija test at 120 deg. F., the Deval test at 170 deg. F., the boiling water test (212 deg. F.), up to the autoclave test where the cement is heated in steam under pressure to temperatures over 300 deg. F.

All accelerated tests for the soundness of cement are open to the criticism that they are unnatural and that cement is never required in practice to withstand the action of hot water, but they have their basis in the fact of experience that cements which do not crack or swell when heated in water or steam for a short time will not expand in practical usage. There has been much controversy over the temperature required for a reliable soundness test. The Faija test of 120 deg. F. held the field for many years, and there is abundant evidence that this was an adequate safeguard against expansion. The Faija test has become obsolete, not on account of its inadequacy, but because well-manufactured cements will withstand higher temperatures, viz., up to the boiling point of water without swelling or cracking.

Boiling water tests in various forms are now the world-wide standard for cement soundness, but there is no agreement as to the best method of application and there is room for improvement in all. In the Le Chatelier test of the British Standard Specification the gauged cement is put in a mould and immersed in cold water immediately, the boiling taking place after twenty-four hours. The expansion is measured by means of pointers fixed to the mould, and is the difference between the pointers immediately before and immediately after six hours' boiling. One of the criticisms of the Le Chatelier test is that some of the expansion takes place while the cement is in cold water, and is not measured. The reply given to this is that such expansion is merely the result of the peculiar effect of water upon cement during setting, which has no relation to the real expansion of the cement. Another defect of the test is that the result depends on the setting time of the cement, so that if a quick-setting cement of high expansion is rendered slow setting the expansion becomes reduced, and conversely if the setting time is expedited the expansion increases. The chief value of this test is that it yields its result in an actual measurement.

The boiling or steaming of pats of set cement avoids the interference due to the action of water on the cement during setting, but there is the disadvantage that the result depends upon the individual observer's judgment of what constitutes " warping and cracking," which are the defects leading to condemnation.

The autoclave or high-pressure steam test is generally considered to be unnecessarily severe in its action on cement, and its adoption would probably lead to the production of cements of decreased strength. The strength and the soundness of cement are in some degree opposing factors, because great strength demands a high proportion of lime in the cement while the greater the proportion of lime the greater is the possibility of failure under severe accelerated tests. Hence, an unduly severe accelerated test may demand a lower proportion of lime in cement, which could only lead to a lower strength.

LARGE CONCRETE BRIDGES.

The suitability of reinforced concrete for bridges of exceptionally large span is shown in the following table. Other bridges which cannot be included as they have not been completed are: one at Balbigny-Règny in the valley of the Bernard Loire, with a span of 570 ft., commenced in 1913 but stopped by the war in 1914 and not resumed; and a bridge of the same span designed by M. Linton for a railway near Stockholm, but not yet built. The largest span in a concrete bridge actually in use is at New York; this bridge has a central span of 720 ft.

The Langwies Viaduct over the Plessur in Switzerland has a span of 300 ft. and is an interesting example of the beautiful effect obtainable with reinforced concrete. As it is at an altitude of 4,400 ft. above sea-level it is subjected to violent changes in temperature, and these are abundantly met by two saw-tooth expansion joints in the structure; notwithstanding its slender form, this bridge its strong enough to sustain locomotives and other vehicles weighing 50 tons each.

DIMENSIONS OF LARGE CONCRETE BRIDGES.

NAME.	SPAN. ft.	WIDTH. ft.	USE
Isar at Grünwald (Germany)	233	27	Road
Gmündertobel at Teufen (Switzerland) .	263	22	Railway
Aare, at Olten (Switzerland).	273	20	Railway
Oere (Sweden). . .	300	17	Railway
Grafton, at Auckland (N.Z.)	325	80	Road
Larimar at Pittsburg (U.S.A.)	305	16	Road
Langwies (Switzerland)	320	70	Road
Rocky (U.S.A.) . .	285	80	Road
Resurrection (Rome) .	333	30	Road
Monroe, Spokane (U.S.A.)	285	17	Road
Halen-sur-Aare (Switzerland).	290	10	Road
Villeneuve (France)* .	327	56	Railway
La Balme sur Rhone (France). . . .	320	30	Road
Franklin on Mississipi (U.S.A.)	2 × 200	87	Road
St. Pierre Rouen (France)	437	87	Road

Rev. du Beton Arme.

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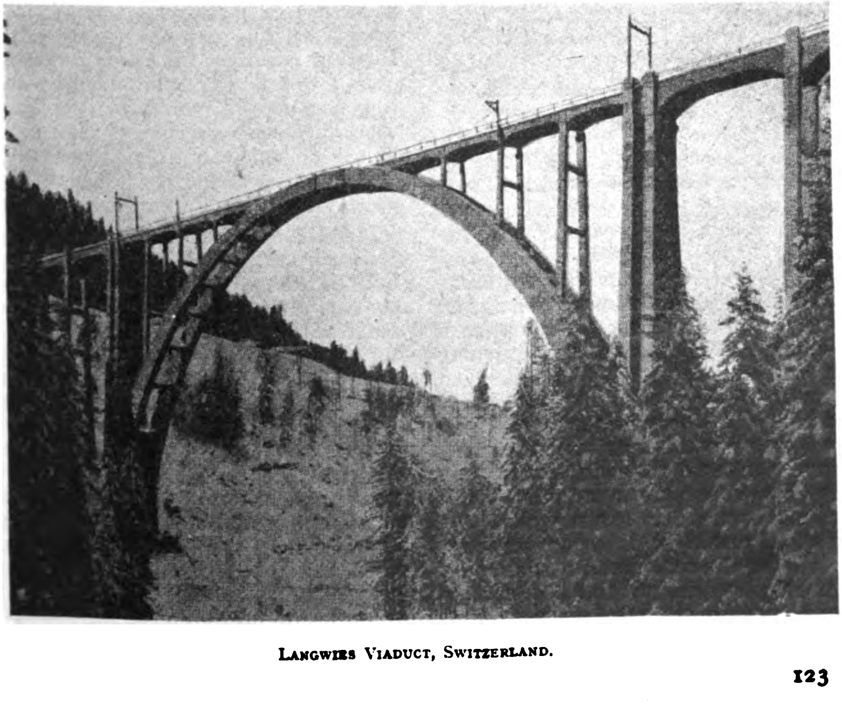
* See *Concrete and Constructional Engineering*, September, 1922.



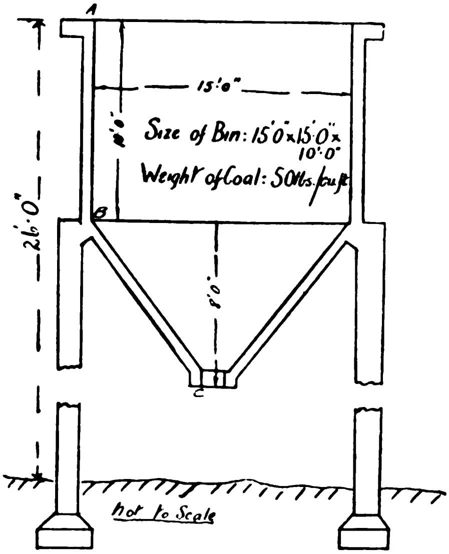
LANGWIES VIADUCT, SWITZERLAND.

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

***Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.*—ED.**

DESIGN OF A COAL HOPPER.

QUESTION.—I should be obliged if you would give me some information as to the nature of the stresses that have to be provided for in a coal hopper of the dimensions shown in accompanying sketch. I have assumed that the portion A—B is a retaining wall, and that the portion B—C is a pyramidal section subject to tension only. Is this correct? I should be pleased to know the nature and amount of the stresses in a hopper of this description filled with coal, weighing 50 lb. per cu. ft. C. E.



ANSWER.—The vertical portion A—B is certainly to be designed to resist internal pressure as a retaining wall, but, of course, the beam at the top, being held at the ends, is not like a retaining wall. It is not correct to assume that the pyramidical part B—C is in pure tension; there is also quite considerable bending on it. A complete design and analysis is, however, much outside the scope of what could be attempted in these columns.

GLAZED CONCRETE SURFACES.

QUESTION.—We understand that a method for covering concrete with a glaze by a cold process has been invented by a German engineer, and we should be glad if you could give us particulars of this process and the name of the inventor.—J. G. & S.

ANSWER.—There are quite a number of German firms who have taken up the manufacture of glazed concrete products, but in the majority of cases the processes for glazing are not giving complete satisfaction. The most successful, perhaps, is that produced by Mr. A. F. Brackmann, of 19, Bessauer Strasse, Lanknitz, Berlin. Instead of treating the surface of the tile only, the tile itself is made of a special mixture, the main part of which is probably a magnesite cement. The coloured mixture is filled into the form, which has a base plate of glass or celluloid; this ensures a smooth surface, and a bright glazing appears during setting. If it is required to colour the surface only, a thin layer of the coloured mixture is placed in the mould and the form is then filled with the uncoloured mixture. It is not claimed that these tiles can withstand heat, weather, or acids, and their use is limited to interior decoration work.

BOOKS ON REINFORCED CONCRETE DESIGN.

QUESTION.—I am anxious to obtain a book on reinforced concrete design which deals with the method of design of horizontal wind bracing on high reinforced concrete structures, such as water towers, supported on a number of columns.—F.W.

ANSWER.—The question is touched on in Dr. Faber's "Reinforced Concrete Design," Vol. I, pp. 214-215. We do not know of any book dealing at all exhaustively on it. "Shearing Resistance of Reinforced Concrete Beams" has not been published yet, but it was to have been based on a series of articles appearing in this Journal between May and November, 1916, and a few issues in 1917.

THE INSTITUTION OF STRUCTURAL ENGINEERS.

THE ARCHITECT AND ENGINEER.

At a meeting of the Institution of Structural Engineers held at No. 296, Vauxhall Bridge Road, S.W.1, on January 25, Mr. W. J. H. Leverton, Lic. R.I.B.A., F.S.Arc., read a paper entitled "The Architect and Engineer."

In the course of his remarks, he said the engineer was utilitarian: his sole object generally was efficient construction. Science was his mistress. If he paid any court to art it was by applying ornament after the form of his structure had been settled, for good or ill, merely by the claims of construction.

The architect in designing his building endeavoured to gain a satisfactory effect by means of good grouping, a fine outline, and a proper disposition of the parts. He then added ornament if the purpose of the building and the purse permitted. The ornament, which was an accessory—a desirable one frequently—was not of course applied indiscriminately, but in the best works emphasised the construction and the important lines of the building. To give an erection architectural character did not by any means involve making it more ornate or costly. It was important to emphasise that because there was an idea that an architect was only necessary when an ornate building was required; that he was an individual who playfully peppered a façade with pilasters and added carving *ad libitum*, possibly *ad nauseam*. There was a fine passage in Viollet-le-duc's lectures, in which he showed that the smallest buildings in Pompeii, built of the cheapest materials, were, owing to their treatment, quite as much works of art as the ornate public buildings of the city.

As architectural treatment did not necessarily involve additional expense, there was no reason why every erection should not possess it and give delight to the beholder. Granting architectural treatment should be given to all erections, how was it to be obtained? By referring to the methods of Sir Christopher Wren, healing the unnatural divorce between architecture and construction (now called engineering), placing the architect in his proper position as general director, and to avoid the troubles through the architect being overburdened with work, he should depute to others such duties of detail as could be devolved without upsetting the success of the scheme.

In order to prevent any misunderstanding which might arise, it would be well to arrive at a clear understanding as to what was meant by an "architect." In his opinion the architect was a person who might be called "The Prime Minister of Building." He should plan and design the general scheme as a grand whole, and have the details worked out by experts in the various branches working under his direction. The principal member of the "Prime Minister's" Cabinet would be the "Minister for Construction"—the engineer—who would settle all the details after the architect had devised the general scheme. As an instance, suppose a wide opening had to be covered—it might be a bridge or an opening in a building. The architect would settle whether a girder or arch should be used. If the opening were in a building the style of architecture, whether trabeated or arcuated, would have to be considered. If the architect decided on a girder the engineer would step in and settle all details as to the number of plates, their sizes, the thickness of web, the number of stiffeners, etc. The architect might have something to say about the depth of the girder, because that might affect the proportion of the opening. If an arch were selected the engineer would settle the depth of voussoirs, the size of abutments, etc., that were necessary for the bare necessities of construction; the architect might possibly add something more if he aimed at a massive monumental effect. The engineer would be the authority for materials, their strength and suitability for the purpose, the bearing capacity of soils, details of retaining walls, piling where required, and other matters too obvious to require mention.

What an impetus it would give to the advancement of architecture if all new methods of construction were handled by men who had been artistically trained. A cry went up at times for a new style. If architectural design were once more based on construction and full advantage taken of new methods something fresh might result. Many of the men of old would have gloried in reinforced concrete. What would the builders of Beauvais have given for such a material ? It would have enabled them to erect a central tower a thousand feet high ! Few of the larger French cathedrals had a central tower that dominated the church like the towers of Canterbury, York, and Lincoln. A central spire for Amiens to hold its own would have to be at least seven hundred feet high. It could be constructed in reinforced concrete with ease, but anything of that height in stone would require supporting piers so enormously large that the centre of the church, the most useful part, would be practically blocked up.

Westminster Abbey badly wanted a central tower and a design modelled on the much-abused western towers was very suitable for erection in reinforced concrete. The existing piers would probably take the additional weight ; they were considered to be too slight for the weight of a stone tower. Mr. Leonard Stokes, PP. R.I.B.A., and Royal Gold Medallist, had carried out a cathedral in the West Indies in reinforced concrete ; the soil of the site would not carry the weight of a stone or brick building.

It was certainly a mistake to hide concrete construction behind a veneer of anything. The concrete building should not be a frame building, a mere skeleton put up and cased afterwards with something else. The piers, instead of being bald squares, should be treated as architectural columns or piers, and the beams as entablatures with cornices cast on properly designed for concrete, not simply stone imitations. A very fine building had recently been built with a cornice in stone of Corinthian type. The modillions were suspended to concealed reinforced concrete cantilevers by dovetailed plugs ; the effect was fine, but one did not feel happy when one knew how it had been obtained.

Referring to business relations between architect and engineer, he said, should the engineer be in partnership with the architect, run his own office, or be an assistant ? There would possibly be room for the three methods. The partnership should work well in many cases. When a structural specialist partner was added to an architect and a business man there would be a triple alliance of satisfactory character. Many of the large American firms had three such partners. In extreme cases it might be desirable to consult an engineer who had specialised in particular work, and he should be paid by the client. In important works clients would know that professional men of various kinds had to be consulted and paid for, but in smaller works the client might consider that he should only pay one man ; then the payment would be made through the architect, who would settle with his partner or assistant as the case may be. Under the present system a scheme for steel construction, or reinforced concrete, might be prepared by an engineering firm, who included the fees for designing in the cost of the work. It would, no doubt, be better for designing fees to be paid for separately ; then the client would know whether he was paying for steel or brains. At present he often thought he was not paying much for the latter and valued them accordingly.

Some might consider he had laid too much stress on the artistic side of the relations and unduly magnified the importance of architecture. They were all, however, citizens first and professional men afterwards ; and were therefore fully agreed that everything erected should be, at least, unobjectionable in appearance. The only way to achieve that desirable result was to have some one in charge who had had proper training in architectural matters. Many of the earlier engineering works were of a high standard—any age might be proud of them. The divorce between engineering and architecture had not fully taken place, for Sir John Rennie was a Fellow of the Royal Institute of British Architects. Some later works, alas, were blots on the fair face of city and country alike. Two Thames bridges might be instanced, Hampton Court and Charing Cross, the former of which was by far the worst. It was assumed in some quarters that because the latter was not what it might be a railway bridge was necessarily ugly, but that was a ridiculous heresy ; a railway bridge might be as fine a structure as a Roman aqueduct.

CONCRETE AGGREGATES—*continued.*

Norfolk—(Watton Carbrooke).

(1) *Source and locality of same :* Norwich Main Road, Watton Carbrooke.
(2) *How obtained :* Digging and screening.
(3) *From whom obtained ?* Mr. W. R. Goulder, Gt. Ellingham Hall, Attleboro', Norfolk.
(4) *Transport facilities :* Not difficult, 2 miles Watton Station, G.E.R.
(5) *Is there any provision at or near source for washing or crushing ?* No.
(6) *Price per cubic yard, and where delivered :* In pit broken to 2 in. to 1¼ in. gauge, 5*s*. 6*d*.
(7) *Is composition uniform ?* Yes.
(8) *Detailed description :—*
    (*a*) *Kind of stone or coarse material :* Flints, require washing.
    (*b*) *Kind of sand or fine material :* Not suitable (brick earthy).
    (*c*) *Size of particles ; approximate percentage that needs crushing to pass ¾ in. screen :* 75 per cent.
    (*d*) *Impurities present* (*as clay, sulphur, coal, organic matter*) *:* Loam, clay, chalk.
(9) *Is sample sent with this sheet :* No.
(10) *General remarks :* Only the flints broken and well washed would be suitable.

Norfolk—(Wereham).

(1) *General description :* Flint gravel.
(2) *Source and locality of same :* Wereham.
(3) *How obtained :* Digging and sifting.
(4) *From whom obtained ?* Wereham Gravel Co.
(5) *Is available quantity limited ? If so, how ?* Only obtainable in small veins.
(6) *Present maximum output per day* (*stated in cubic yards*) *:* 20.
(7) *Transport facilities :* G.E.R. 3 miles.
(8) *Is there any provision at or near source for washing or crushing ?* No.
(9) *Price per cubic yard, and where delivered :* 4*s*. 6*d*.
(10) *Is composition uniform ?* No.
(11) *Detailed description :—*
    (*a*) *Kind of sand or fine material :* Flint siftings.
    (*b*) *Relative proportions of coarse and fine material :* 80 coarse, 20 fine.
    (*c*) *Shape of particles* (*rounded, angular, flat, etc.*) *:* Rounded.
(12) *Is sample sent with this sheet ?* No.

Norfolk—(Widow's Hill, Stanford).

(1) *Source and locality of same :* On road from Tottington to Stanford.
(2) *How obtained :* Digging and screening.
(3) *From whom obtained ?* Merton Estate Offices, Watton, Norfolk.
(4) *Is available quantity limited ? If so, how ?* Worked by various local authorities.
(5) *Transport facilities :* 7 miles station (Watton, G.E.R.).
(6) *Is there any provision at or near source for washing or crushing ?* No.
(7) *Price per cubic yard, and where delivered :* In pit broken to 2 in. to 1¼ in. gauge, 5*s*. 6*d*.
(8) *Is composition uniform ?* Yes.
(9) *Detailed description :—*
    (*a*) *Kind of stone or coarse material :* Clean flint.
    (*b*) *Kind of sand or fine material :* Clean flints.
    (*c*) *Relative proportions of coarse and fine material :* Almost any size from 2 in. to ½ in.
    (*d*) *Shape of particles* (*rounded, angular, flat, etc.*) *:* Rounded.

(*e*) *Size of particles : approximate percentage that needs crushing to pass ¾ in. screen :* 70 per cent.
(*f*) *Impurities present* (*as clay, sulphur, coal, organic matter*) *:* Little or none.
(10) *Is sample sent with this sheet :* No.
(11) *General remarks :* Good gravel pit, but transport difficult. No road to pit for traction haulage. Carting very costly.

Norfolk—(Wymondham and Hackford).

(1) *General description :* Shingle, stones, and sand.
(2) *Source and locality of same :* Wymondham and Hackford.
(3) *How obtained :* Pits.
(4) *From whom obtained ?* Mr. C. R. Ayton, Wymondham ; Mr. S. L. Semmence, Wymondham ; Mr. I. W. Wavey, Hackford.
(5) *Present maximum output per day* (*stated in cubic yards*) *:* 100 yds.
(6) *Transport facilities :* Rail and road.
(7) *Is there any provision at or near source for washing or crushing ?* No.
(8) *Price per cubic yard, and where delivered :* 4*s.* 6*d.* in pit.
(9) *Is composition uniform ?* Yes.
(10) *Detailed description :—*
  (*a*) *Kind of stone or coarse material :* Flints, clean.
  (*b*) *Kind of sand or fine material :* Sand and shingle.
  (*c*) *Relative proportions of coarse and fine material :* Stone and 2 to 1 sand plentiful.
  (*d*) *Shape of particles* (*rounded, angular, flat, etc.*) *:* Round and angular.
  (*e*) *Size of particles ; approximate percentage that needs crushing to pass ¾ in. screen :* All sizes.
  (*f*) *Impurities present* (*as clay, sulphur, coal, organic matter*) *:* Clay (if any).
(11) *Is sample sent with this sheet ?* No.

Norfolk and Suffolk.

(1) *General description :* Ballast pits.
(2) *Source and locality of same :* Homersfield, Suffolk.
(3) *How obtained :* Gravel workings.
(4) *From whom obtained ?* Mr. J. Edwards, contractor, Norwich.
(5) *Is available quantity limited ? If so, how ?* 1,000,000 yds. or more.
(6) *Present maximum output per day* (*stated in cubic yards*) *?* No.
(7) *Transport facilities :* Horses.
(8) *Is there any provision at or near source for washing or crushing ?* Not at present, but water present below a certain level in pits.
(9) *Price per cubic yard, and where delivered :* On rail Homersfield G.E.R. ; dependent on any screening required.
(10) *Is composition uniform ?* Yes.
(11) *Detailed description :—*
  (*a*) *Kind of stone or coarse material :* Flint.
  (*b*) *Kind of sand or fine material :* Sharp silica.
  (*c*) *Relative proportions of coarse and fine material :* Good natural aggregate.
  (*d*) *Shape of particles* (*rounded, angular, flat, etc.*) *:* Irregular.
  (*e*) *Size of particles ; approximate percentage that needs crushing to pass ¾ in. screen :* Say, 40 per cent.
  (*f*) *Impurities present* (*as clay, sulphur, coal, organic matter*) *:* Virtually nil.
(12) *Is sample sent with this sheet ?* No.
(13) *General remarks :* Have supplied quantities just as raised for aerodrome and building construction.

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	6
Clean Shingle, ¾ in. mesh	,,	14	6
,, ,, ½ in. mesh	,,	15	6
Best British Portland Cement	per ton 58*s.* to	63	0
"Ferrocrete" Portland Cement	10s. per ton extra.		

Boarding for Shuttering—		*Sawn.*		*Wrot.*	
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

Sawn Timber for Struts and Supports—	
3 in. by 4 in.	from £20 per standard.
3 in. by 6 in. and 3 in. by 7 in.	,, £22 ,,

Mild Steel Rods for Reinforcement—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	6
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds.	,,	14	6
¼ in. Rounds.	,,	16	6

MATERIAL AND LABOUR INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

Portland Cement Concrete well mixed with Clean Water in proportion.

1 : 2 : 4—				s.	d.
Do.	do.	in foundation	per yard cube	45	0
Do.	do.	in columns	,, ,,	50	0
Do.	do.	in beams	,, ,,	50	0
Do.	do.	in floor slabs 4 in. thick	per yard super	5	3
Do.	do.	in floor slabs 6 in. thick	,, ,,	7	9
Do.	do.	in floor slabs 9 in. thick	,, ,,	11	3
Do.	do.	in walls 6 in. thick	,, ,,	8	0

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

Steel Reinforcement, including Cutting, Bending and putting into Position and securing with Stout Binding Wire—		s.	d.
From ¼ in. to ⅜ in.	per cwt.	26	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

Shuttering—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	60	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	50	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11
Do. do. as last in narrow widths	,, ,,	1	1
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1
Do. do. as last in narrow widths	,, ,,	1	3
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

MEMORANDA.

The Institution of Engineering Inspection.

THE Institution of Engineering Inspection has been formed to act as the responsible authority concerned with the development of skilled inspection, by setting a standard of ability for technical inspectors, by looking after the professional interests of those who specialise in this work, and by enabling merchants and producers to be in touch with those who can help them to secure and maintain a high quality in their commodities. Mr. N. P. P. Sandberg, M.Inst.C.E., is the President, and Messrs. F. L. Galloway, C.M.G., and Francis R. Wade, M.Inst.M.E., M.Inst.Struct.E., Vice Presidents. The office of the Institution is at Palace Chambers, S.W.1.

Building in Winter.

FURTHER to the Editorial in our last issue, it is interesting to note that an extension of the building season in order to reduce unemployment among workers engaged in the industries concerned is being urged by the Builders' Exchange of Montreal. An official of the Exchange states: "There are periods of the year when a scramble for labour and materials takes place, forcing up the cost of both and hampering building operations, followed later by a winter period during which materials are not wanted and building workers can find nothing to do. The object of the present movement is to even up the building season so that there will never be a serious rush for men and materials nor a period of complete idleness."

Relative Costs of Concrete and Steel.

THE consulting engineers to the Kirkcaldy Town Council have submitted schemes for enlarging the berthing facilities at Kirkcaldy Dock by straightening out the bend in the east quay wall, and thus making room for the berthing of an additional vessel. Scheme 1 consists of a timber and steel construction set into the rock of the dock bottom, with flat-bottomed rails spiked level into a wooden decking to carry waggons and cranes, the cost to be about £3,900. Scheme 2 consists of concrete piers on the rock and carrying concrete arches, with concrete front wall and cope, concreted over to the quay level and with rails laid flush with the surface, the cost to be about £2,750. Scheme 3 is for a construction of reinforced concrete entirely, braced to the existing quay, with decking of concrete and rails embedded flush with the quay, the cost to be about £2,100. The engineers and the Harbour Committee have recommended the Council to proceed with Scheme 2.

Port Works at Guayaquil.

THE Port Authorities at Guayaquil (Ecuador) have asked for tenders for the construction of a dock and Custom-house. Specifications call for the use of reinforced concrete, a wharf with up-to-date loading and unloading facilities, the construction of dock warehouses and Custom-house, as well as offices for the use of the Customs guards, the captain of the port, and the Maritime Public Health Service. Other contracts to be entered into include the erection of repair shops and supply sheds. The new dock will have a wharf with a capacity for docking three 10,000-ton vessels at one time.

The Aber-Vrac'h Tidal Power Scheme.

THE French Chamber has passed a Bill for the creation of an experimental tidal electric power station at Aber-Vrac'h (Finisterre). The proposal is to construct a barrage 492 ft. long across the estuary, the top of the barrage being arranged to come 13 ft. above the spring tides high-water mark. The barrage is to be constructed of a number of reinforced concrete caissons. The centre caisson, which is to be 114 ft. 9 in. long and 72 ft. broad, will contain the generating machinery, while on the right-hand side of the barrage a lock will be constructed for the passage of fishing craft.

New Reinforced Concrete Quay Wall.

MESSRS. SHELL MEX, LTD., have decided to replace the timber quay wall at their Lensbury Dock with a reinforced concrete structure to the design of Mr. R. N. Stroyer, specialist in sea and river work, of 4, Westminster Palace Gardens, S.W.1. The contract for the work has been placed with Messrs. Perkins & Son, Ltd., Bristol.

PROSPECTIVE NEW CONCRETE WORK.

Barnoldswick.—*Road.*—The Ministry of Transport has approved the scheme for a new road between Barnoldswick and Kelbrook. The estimated cost is £96,409.

Bradford. — *Road.* — The Ministry of Transport has sanctioned the construction of a new road from Odsal Top to Rooley Lane. The cost will be over £70,000.

Brighton.—*Tennis Courts.*—The T.C. has under consideration a proposal to construct hard tennis courts on the sea front at a cost of £600.

Burslem.—*Sewage Works.*—The Engineer to the Stoke-on-Trent Corporation has received instructions to carry out the following sewage works:—Extensions at the Burslem Works, £19,300; Meir sewage disposal scheme, £15,000; and works at Hanley in connection with the River Trent improvements, £6,000.

Conisborough. — *Sewage Works.* — The Ministry of Health has given its sanction to the U.D.C. to borrow £6,300 in connection with the sewage works extension.

Epsom.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application of the U.D.C. for sanction to borrow £15,000 for sewage disposal works.

Fraserburgh.—*Esplanade, etc.*—The T.C. proposes to construct an esplanade at the Beach, at an estimated cost of £6,600.

Glasgow.—*Boulevard.*—The Corporation has adopted a scheme for the construction of a boulevard, about 4½ miles long and 140 ft. wide, between Anniesland Cross and Duntocher, at an estimated cost of £500,000.

Gosport.—*Bridge, etc.*—The U.D.C. has approved the construction of a new ferro-concrete bridge, 30 ft. wide, across the Workhouse lake, at an estimated cost of £7,000.—The Borough Engineer has prepared plans for the construction of concrete bathing huts, together with refreshment rooms, on the beach at Stokes Bay, at an estimated cost of £1,750.

Great Yarmouth.—*Defence Work.*—The Borough Engineer has been instructed to prepare a plan and estimate for a concrete wall along the Lower Parade.

Kirkcudbright.—*Bridge.*—The Southern District Committee of the C.C. is considering a proposal to construct a new bridge over the Dee at Kirkcudbright at an estimated cost of £33,000.

Leamington.—*Tennis Courts.*—The T.C. has decided to construct six additional hard tennis courts in Victoria Park at a cost of £616.

Lowestoft.—*Sea Wall.*—The Corporation has decided to construct a sea wall, at an estimated cost of £94,000.

Manchester.—*Roads.*—The Corporation Town Planning Committee and the Chadderton U.D.C. have decided to proceed shortly with the construction of two new roads on the north side of Manchester. The estimated total cost is £270,000.

Norwich.—*Bridge.*—The Corporation proposes to construct a new reinforced concrete bridge in place of the present Whitefriars Bridge, at an estimated cost of £14,932.

Norwich.—*Harbour Works.*—The T.C. has decided to construct 100 ft. of quay at Riverside, at an estimated cost of £1,000.

Ossett.—*Water Tower.*—The Water Committee of the U.D.C. has decided to invite tenders for the erection of a water tower.

Oxford.—*Bridges.*—The Engineer to the Thames Conservancy has received instructions to prepare plans for two proposed bridges across the Thames at Oxford.

Pentewan.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the St. Austell R.D.C. for sanction to borrow £3,742 for sewerage and sewage disposal works at Pentewan.

Plymouth. *Open-air Baths.*—The Ministry of Health has sanctioned a loan of £18,300 for the construction of two open-air swimming baths at Bullock's Dock, Richmond Walk.

Purfleet.—*Road.*—It is stated that the Ministry of Transport proposes constructing a new road between Purfleet and Tilbury at a total cost of £200,000.

Queensferry.—*Bridge.*—The Flintshire C.C. has decided to reconstruct Queensferry Bridge.

Ramsgate.—*Drive.*—The Borough Surveyor has received instructions to prepare plans and an estimate for constructing an undercliff drive on the West Cliff estate.

St. Stephens. — *Sewage Works.* — The Ministry of Health has held an inquiry into an application by the St. Austell R.D.C. for sanction to borrow £5,020 for sewerage and sewage disposal works for the parish of St. Stephens (in Brannel).

St. Just.—*Road.*—The U.D.C. has decided to construct a new road between Bojewyan and Keigwin.

Sheffield.—*Road.*—The Corporation proposes to construct a new road between Intake and Gleadless at an estimated cost of £40,000.

Shrewsbury. — *Bridge.* — The T.C. has decided to proceed with the reconstruction and widening of the English bridge across the river Severn. The estimated total cost is £70,000.

Stranraer.—*Road and Concrete Wall.*—The T.C. has decided to construct a new road, from Lewis Street to Glebe Street, at an estimated cost of £5,500. The Council has also agreed to build a concrete wall along Cairnsvan Road at a probable cost of £948.

CYLINDERS of STEEL SHEET PILING

The use of a comparatively light section of Steel Sheet Piling for the construction of cylinders has many advantages, such as **Low First Cost, Great Saving in Freight** as the sections are shipped loose and assembled on the site. **Great Strength** due to **Rigid Interlock** and large number of vertical ribs (flanges of the sections), which on the inside are embedded in the concrete and form a strong reinforcement. They can be **Driven Very Rapidly** to any desired level and added to if required by threading on additional lengths. For work where **Heavy Loads** and **Large Spans** have to be taken " **Simplex** " **Steel Piling Cylinders** are the **Cheapest** and the **Best.**

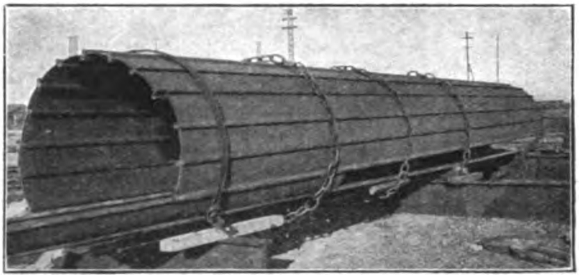


Illustration shows cylinder 45 ft. long constructed of 8-in. **Simplex** Piling, weighing 22 lbs. per square foot, assembled at our works.

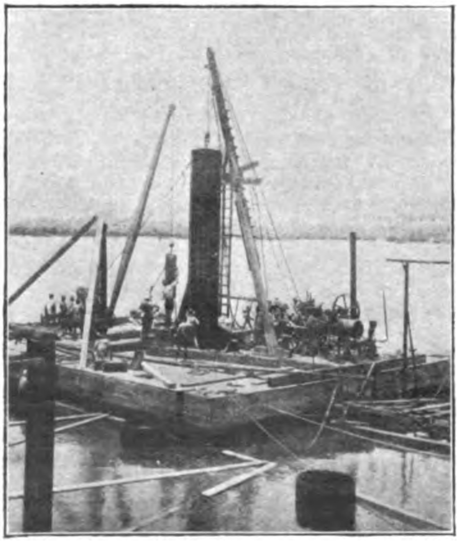


Illustration shows these cylinders re-assembled and being driven on the job with a No. 6 **McKiernan-Terry** Hammer.

Note.—Completed cylinders in foreground.

The BRITISH STEEL PILING CO.

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BILLITER ST.,
LONDON, E.C.3.
Works:
Claydon, Suffolk.

SURREY.—*Roads.*—The Surrey C.C. has agreed to co-operate with the Ministry of Transport and the L.C.C. in the construction of two new arterial roads. The estimated total cost is £490,000.

TYWARDREATH. — *Sewage Works.* — The Ministry of Health has held an inquiry into an application by the St. Austell R.D.C. for sanction to borrow £12,060 for sewerage and sewage disposal work in the parishes of Tywardreath and St. Blazey.

WIMBLEDON.—*Concrete Walls.*—The T.C. has decided to apply for sanction to borrow £5,161 for the erection of reinforced concrete walls at Wimbledon Park.

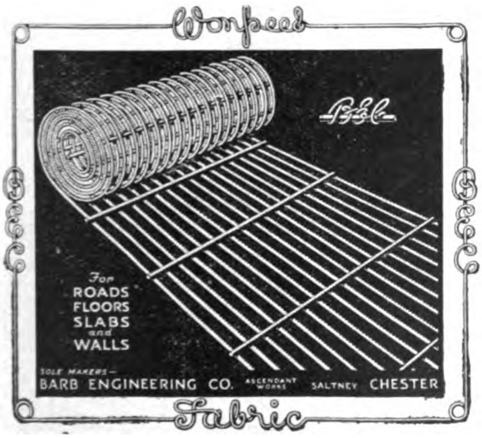
WINCHESTER.—*Bridge.*—The City Council has adopted a report of the Main Roads Committee recommending the erection of a steel and reinforced concrete bridge. The estimated cost is £2,299.

WREXHAM.—*Open-air Bath.*—The T.C. has decided to provide an open-air swimming bath and slipper baths at the Wellow depôt.

NEW COMPANIES REGISTERED.

TRIANGULAR CONSTRUCTION CO., LTD. (185,485). Registered November 3, 1922. To acquire and take over as a going concern and carry on the business known as the Triangular Concrete Construction Co. and Imber Court Engineering Works, now carried on by Major W. H. Smith at Imber Court, Thames Ditton. Nominal capital, £55,000 in 55,000 shares of £1 each. Directors: Lt.-Col. W. Pepys, Cavalry Club, London; Major W. H. Smith, Imber Court, Thames Ditton; C. F. D. Marshall, Clinthurst Lodge, Guildford; C. S. Payne, Imber Court, Thames Ditton; G. J. Davis, The Old Manor House, East Molesey. Qualification of directors: £500.

STANDARD RE-INFORCED CONCRETE ENGINEERING CO., LTD. (185,176). Registered October 20, 1922. To carry on the business of reinforced concrete designers and engineers. Nominal capital, £3,000 in 3,000 shares of £1 each. Directors: F. Majer, 3, Webb Drive, Undercliffe, Bradford; H. Murgatroyd, 13, Marlborough Road, Manningham, Bradford. Qualification of directors: £1; remuneration of directors: to be voted by company.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The latest invention for saving time, labour, the most precise solution of Bending and Direct Stress.

One Minute instead of One Hour.

AGENTS:—BRITISH INDIA: K. S. DHONDY & Co., Byculla Bridge, Bombay. EGYPT: ING. V. R. COMAS & MD. AWADALAH, Alexandria, 11, Place Mohamed Aly. FRANCE: SOCIÉTÉ WATPROOF, 62 rue St. Lazare, Paris. BELGIQUE: V. MUZAK, Ingénieur des Ponts et Chaussées, Bruxelles, 50, rue de la Loi.

INVENTOR: PROFESSOR RIEGER, Polytechnic High School, Brno, Czechoslovakia.

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A NEW MODEL

The "VICTORIA" Self-Contained Portable Mixer — Model No. 7.

THE new improved type of Victoria Concrete Mixer illustrated on this page should prove of interest to every contractor and user of concrete. This new model is driven by a 5-h.p. petrol engine, and is mounted on road wheel truck, complete with side loader and water tank. It is entirely self-contained and has an exceptionally large opening on the filling side which facilitates loading and makes inspection easy. The large mixing drum is one of its special features, and the machine is capable of an output of about 60 to 80 cubic yards per day. Full particulars of this latest Stothert & Pitt model will gladly be sent on request. Why not write us to-day?

STOTHERT & PITT LTD
MIXER DEPARTMENT
ORCHARD STREET WESTMINSTER S.W.1

TENDERS ACCEPTED.

Chapel-en-le-Frith. — *Reservoir.* — The R.D.C. has accepted the tender of Mr. C. Wade, of Luddendenfoot, Yorks, at £5,354 2*s.* 8*d.*, for the construction of a concrete service reservoir, and other work in connection therewith.

Epsom.—*Sewer, etc.*—The Epsom U.D.C. has, provisionally, accepted the tender of Messrs. H. C. Pullar & Co., at £13,273 12*s.*, for the construction of a 12-in. diverting sewer, mass concrete tanks, etc.

Glasgow.—*Bridge.*—The T.C. has accepted the tender of Messrs. Melville, Dundas & Whitson, Glasgow, at £4,655, for the erection of a reinforced concrete bridge at Cumbernauld Road.

Greenwich.—*Coal Bunkers.*—The L.C.C. has accepted the tender of Messrs. D. G. Somerville & Co., Ltd., at £1,893, for supplying and laying reinforced concrete layer to cover the inner surface of coal bunkers at Greenwich power station.

Hadleigh (Suffolk).—*Bridge.*—The West Suffolk County Council has accepted the tender of Messrs. Wakelin & Rampling, of Great Cornard, near Sudbury, at £449 9*s.* 5*d.*, for the construction of a reinforced concrete culvert 4 ft. diameter, and concrete retaining walls in connection with the reconstruction of Red Hill Bridge.

Minehead. — *Reservoir.* — The Minehead U.D.C. has accepted the tender of Messrs. C. V. Buchan & Co., Ltd., of Brighton, for the construction of a covered-in reinforced concrete reservoir capable of holding one million gallons.

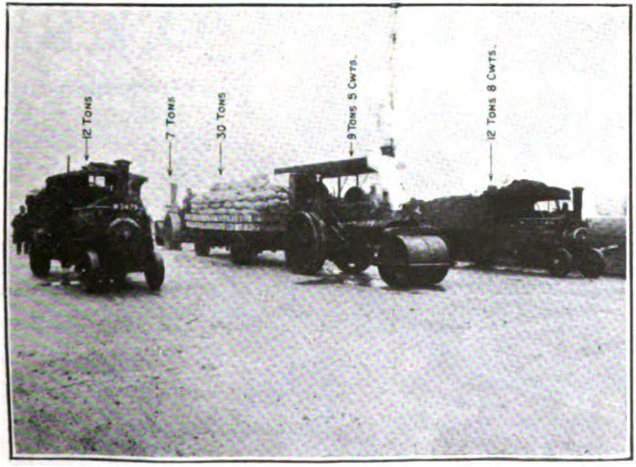
Rochdale.—*Concrete Sewer.*—The T.C. has accepted the tender of Messrs. Radford, Tebbs & Co., Ltd., of Derby, for laying a reinforced concrete sewer in Oldham Road.

St. Peter Port.—*Concrete Deck.*—The Board of Administration has accepted the tender of Messrs. Chessums, Ltd., of Tottenham, London, at £987, for the removal of the timber deck at the North Pier head, and the construction of a new deck in reinforced concrete at the Harbour of St. Peter Port, Guernsey.

Swansea. — *Reservoir.* — The Swansea R.D.C. has accepted the tender of Messrs. Watson & Horrocks, of Bridgend, at £5,803 3*s.* 6*d.*, for various works, including the construction of a concrete service reservoir at Penllergaer.

CONCRETE ROAD TEST IN MANCHESTER.

The widest arterial road in reinforced concrete now being laid by the Manchester Corporation, between Moston Lane and Rochdale Road, was subjected to test last month. As fully described in our issue for November last, the road was laid in alternate bays of 55 ft. in width by 13 ft. in length, and reinforced by the Walker-Weston interlocked double-layer framework. In view of the satisfactory results of the tests, the construction of this road over the remainder of the peat bog is being proceeded with in exactly similar manner to the portion of the road already laid. The photograph shows the test load. The vehicles travelled in a line from one end of the road to the other.



Test on Manchester Arterial Road.

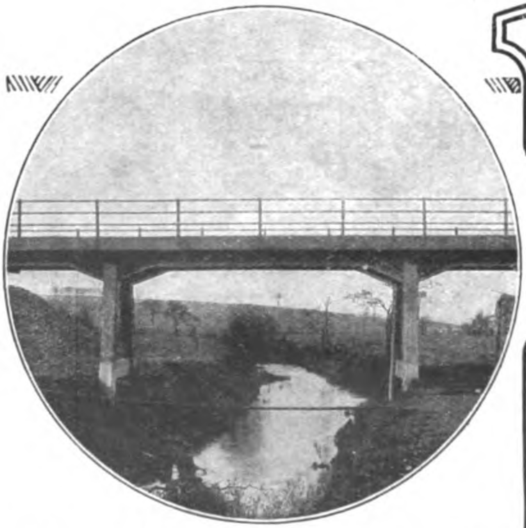
TRADE NOTICES.

THE name of The Considere Construction Co., Ltd. (72, Victoria Street, S.W.1), has now been changed to "Considere Constructions, Ltd." This has been done with a view, as far as possible, of avoiding any title which might give the idea that the Company was actually engaged in contracting work, its business (as for the past fifteen years) being entirely confined to the detail calculations and design of reinforced concrete. M. Pelnard-Considere and M. Caquot, of the French Firm of Pelnard-Considere, Caquot et Cie., have joined the English Board of Directors.

Messrs. Johnson's Reinforced Concrete Engineering Co., Ltd., have removed their London office from No. 1, Basinghall Street, E.C.2, to No. 23, Leadenhall Street, E.C.3 (Telegraphic address: Crossways Fenn, London; Telephone: Avenue 1680-1).

Sir William Noble, formerly Engineer-in-Chief to the General Post Office, has joined the Board of Directors of the Walker-Weston Co., Ltd., reinforced concrete engineers, of No. 7, Wormwood Street, London, E.C. Sir William Noble is Chairman of the Broadcasting Committee which inaugurated the Broadcasting Company, and a Director of the General Electric Company.

Some interesting additions have been made by Messrs. Winget, Ltd., of Grosvenor Gardens, S.W.1., to their extensive range of concrete machinery in the form of three new mixers and a new block-making machine. Two of the mixers, namely, the semi-spiral and the chain spade types, are of 4 cu. ft. capacity, while the "Warwick" type, of 2½ cu. ft. capacity, should be particularly useful in supplying the demand for a cheap and simple machine which has no complicated parts to get out of order, and is exceptionally light and easy to transport from job to job. All the mixers are mounted on wheel-trucks and driven by petrol engines. In the "semi-spiral" type the materials travel upwards to the discharging end, thus obtaining a low charging height, and a discharging level suitable for depositing the mix straight on to the site where it is required or into barrows. The new "No. 3." pressure machine is a well-made and efficient plant for forming blocks 18 in. by 9 in. by from 2 in. to 4½ in. in thickness, or blocks 24 in. by 12 in. by from 2 in. to 3 in. in thickness. Illustrated folders relating to the different machines will be sent on application to Messrs. Winget, Ltd.



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CONCRETE
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CONSTRUCTIONAL ENGINEERING

MARCH **1923** **VOL. XVIII. No. 3**

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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No. 184

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By Appointment to
H. M. King George V.

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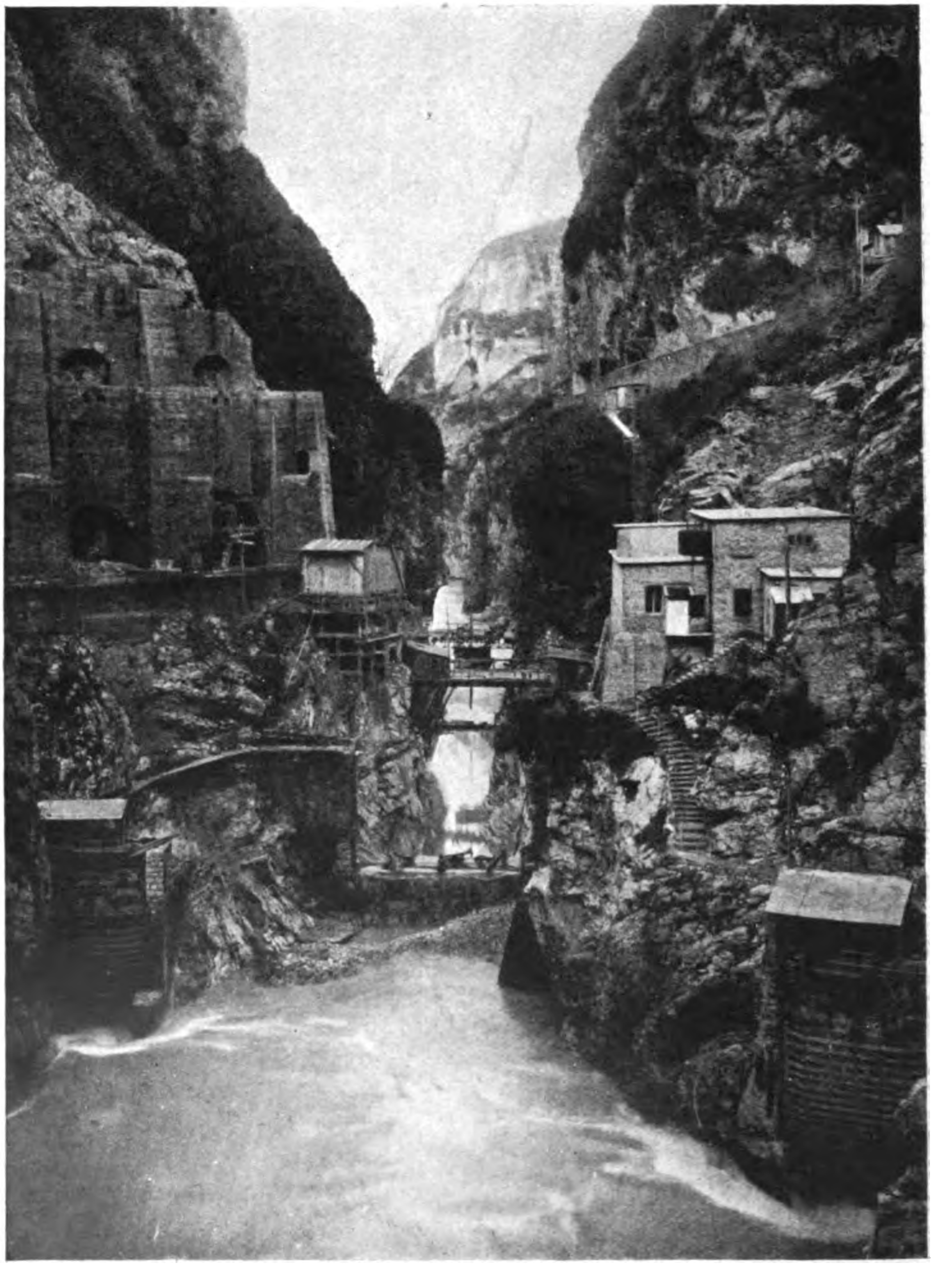

THE DAM AT THE PORTES DU FIER: GENERAL VIEW SHOWING PRELIMINARY WORKS.

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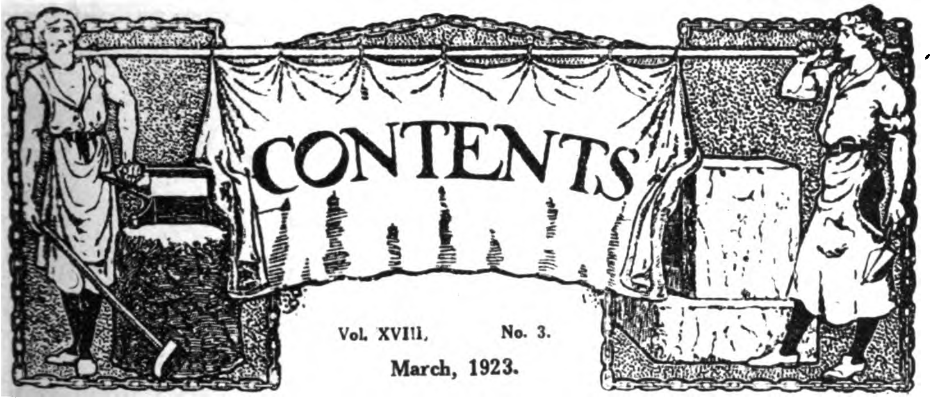

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Let us quote YOU for this the best small Mixer

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THIS Trade Mark typifies Knowledge, Sound Judgment and Wisdom, and suggests the wisdom of specifying RANSOME machinery, "the reliable make that LASTS."

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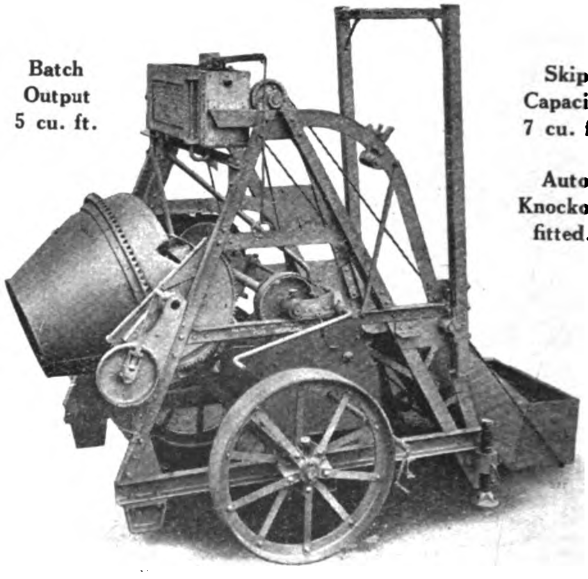
which gives full details and photographs of the 1923 Ransome Lightweight Mixer.

If interested in larger sized Mixers ask for Catalogue No. 151, and Hoisting and Placing Equipment Catalogue No. 16.

Ransome Lightweight Concrete Mixer.



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Auto Knockout fitted.

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CONCRETE
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Volume XVIII. No. 3. LONDON, MARCH, 1923.

EDITORIAL NOTES.

THE ARCHITECT, THE ENGINEER, AND ANCIENT MONUMENTS.

IN his paper read at the Institution of Structural Engineers recently, an abstract of which we gave in our last issue, Mr. Leverton raised a much-debated question when he discussed the relations between the architect and the engineer. On the general question we do not intend to dwell, except to say that on works of any magnitude the two professions are quite indispensable one to the other and should work in the closest collaboration from the very beginning of the enterprise. In the discussion on the paper (which we report on another page), however, the question of the employment of the engineer in the preservation of ancient monuments was again raised. Tintern Abbey and Westminster Hall were mentioned as awful examples of what happened when such work was left to engineers, and a point was made of the fact that " no attempts had been made to restore the old stone-work, but steel girders had been employed," although " it was true that not a bit of the steel-work could be seen." Such a statement raises the whole question of the object we have in view in taking measures to prevent what remains of our ancient buildings from entirely falling to pieces. It seems to us that the words " restore " and " restoration " are not at all applicable to the object in view, which should be to keep the buildings for posterity as much as possible in the same condition as they were when originally built. To deplore the fact that no attempts have been made to " restore " the old stonework which has crumbled or fallen away is to suggest that such work should be replaced with new stone, but do those who favour that procedure fully realise whence it will lead ? With the passage of time it must inevitably mean that not one of the original stones will be left standing upon another. It will, in fact, result in future generations seeing not a building which has become a landmark in our history, or to which cherished traditions cling, but a new building of the same design re-erected on the same site. It seems a much better word to use in this connection is "preserve," and it must be admitted that if this is our object the method adopted by H.M. Office of Works of reinforcing the structures with steel girders and reinforced concrete is much better than patching the fabric with new stone. The statement that " not a bit of the steel-work can be seen," seems to us to be the very best argument in its favour. No one, if he had to wear an old coat, would prefer one with patches rather than one on which the " invisible mender " had displayed his art, and the principle can well be applied to old buildings. If it is desirable that they should be preserved at all, surely it is better that they should be left as far as possible in the same state as when they were taken in hand than that their decrepitude should be emphasised by shoring or that they should be practically rebuilt, for it is their aged and weather-beaten appearance which is their charm. These buildings have already suffered too much at the hands of the "restorer"; our object now should be to preserve.

ECONOMICS IN WATER SUPPLY.

ALTHOUGH the saving in the cost of water supply and sewerage works made possible by the use of concrete pipes is now very generally appreciated, and is reflected in their extended adoption, it is not often such striking figures are given as those detailed by Mr. J. C. Ross, M.Inst.C.E., in a paper to be brought forward for discussion at the Institution of Civil Engineers. The paper deals with the improvement of the water supply to the city of Hobart, Tasmania, in the works connected with which a great deal of reinforced concrete work has been used. The reference to the water mains is of particular interest, and we cannot do better than quote the author's own words: "In order to supply Ridgeway reservoir," he says, "a new main was laid from the headwaters of the North-West Bay River on the slopes of Mount Wellington—a distance of about nine miles, of which about one-half was laid with 15-in. piping and the other half with 18-in. piping. This pipe-line and the discharge-pipe from Ridgeway to the distributing basin, also 15-in. diameter, as well as the mains from the distributing basin to the lower reservoirs and city connections (a total length of about 10½ miles), were intended to be cast-iron or steel; but owing to the high prices then ruling they were laid instead with centrifugally-made reinforced-concrete pipes, which have proved thoroughly satisfactory. The 15-in. and 18-in. supply mains were designed and tested to 75 lb. per sq. in. when laid, and the 15-in. line (from the reservoir to the distributing basin) was designed and tested to 130 lb. per sq. in. The substitution of concrete for iron pipes effected a saving of more than 60 per cent., the lowest price for iron being £28,824 exclusive of the cost of cartage and laying, while the concrete line cost £13,624 including the cost of cartage and laying." It does not need any comment to emphasise such a statement of fact, but we would specially like to point out to those responsible for putting work in hand for the relief of unemployment the advantages of adopting the cheapest methods when they are, in Mr. Ross's words, also "thoroughly satisfactory." For not only does the use of concrete make more employment locally, but if the work can be done 60 per cent. cheaper, it follows that the expenditure of a fixed sum will produce more than double the amount of work and also provide employment for more than double the number of people. It is a fallacy often met with when the efforts of a local authority to relieve unemployment are judged by the amount of money it spends on "relief works," for nothing is farther from the truth when such undertakings necessitate the importation of considerable quantities of manufactured goods from another district. The only way to make the best use locally of money allocated to the provision of work for the relief of distress is to make quite certain that the bulk of it will go to local labour, and it can be safely said that no material offers more scope in this direction than concrete, whether the work under consideration be a pipe-line, a new road, a building, or any other constructional work.

THE LIFE OF CONCRETE SEWERS.

First cost, however, is not always an accurate criterion of the cheapness or otherwise of any commodity. Cheap articles which require frequent renewal or continual expenditure to keep in repair generally prove to be false economy, and ultimately more expensive than a first-class article which, although dearer to buy in the first instance, necessitates but little or no expenditure during a longer period of usefulness. Since writing the above note we have received an account of the experience of Mr. J. W. Tomlinson, A.M.Inst.C.E., Borough Surveyor of Luton, as to the relative merits of iron and concrete pipes from the point of view of longevity and maintenance costs. The instance is of more particular interest because in a concrete sewer just completed in that town a glazed brick lining was put in at the bottom for fear of any disintegration of the concrete which might be caused by the acids in the sewage. Last month the Surveyor inspected a concrete sewage pipe, one mile long, through which sewage is pumped with a 75 ft. head, laid eleven years ago, and through which 3,000,000 gallons of sewage are pumped every twenty-four hours. To inspect the concrete pipe it was necessary to pass through an iron pipe laid at the same time, and it was found that although the iron pipe was badly pitted and rough, the concrete pipes were quite smooth, except for the marks of the shuttering used when they were built. The concrete surfaces were tested with a chisel, and it was found that there was no disintegration whatever. The Surveyor has since stated that had he made this inspection before laying the pipe just completed, he would have used concrete throughout and dispensed with the glazed bricks. These two instances prove that cheapness in first cost is not the only claim which can be put forward for concrete sewage pipes. When a low initial cost is combined with long life and no maintenance cost whatever it would seem that the ideal has been reached.

ROADS.

The enormous increase in the use of mechanical transport during recent years has naturally led to a great deal of attention being given to the problems of making existing highways suitable for carrying the traffic imposed upon them and of building new roads. Vast sums of money are now being, and for many years must be, spent on roads in order to meet the new conditions, and it is a matter of vital national importance that this money should be spent to the best advantage. We have frequently pointed out the phenomenal manner in which the all-concrete road has come into popular favour in America, Canada, and elsewhere, but it is not, we think, generally realised that nearly two-hundred all-concrete roads have been laid in this country since the end of the war, when the number was less than twenty. This growth indicates that many Borough Surveyors and others hold the opinion that the all-concrete road has fully justified the claims made for it as the cheapest form of road construction, and, further, that the roads already laid in this country (some as long as twenty years ago) have proved its wear-resisting properties. In view of the importance of the matter, we propose to devote part of our next issue to illustrations and articles on this type of road construction.

THE DESIGN OF STEEL-FRAME BUILDINGS.

A DISCUSSION WITH PARTICULAR REFERENCE TO THE JOINT CONFERENCE REPORT.

By **EWART S. ANDREWS, B.Sc. Eng., A.M.Inst.C.E., M.I.Struct.E.**

INTRODUCTORY AND HISTORICAL.

THE design of steel-frame buildings in the London area is governed by the L.C.C. (General Powers) Act, 1909. Prior to this Act steel-frame buildings were not allowed in London, or rather they were allowed only if the brick walls were thick enough to carry the loads without the assistance of the steel frame. Although the Act has legislative authority in London only, the fact that it has received Government blessing and contains, I believe, the only official rules for these buildings, has the effect that it is adopted by a large number of local authorities in this country and in the Colonies.

Experience has shown that the Act is unduly severe on design in some respects, and that there have been differences in interpretation of various clauses on the part of District Surveyors whose duty it is to administer the Act. Some years ago, therefore, a Conference was convened with a view to securing uniformity of practice; this Conference was composed of representatives of the Concrete Institute, The District Surveyors' Association and the Royal Institute of British Architects, and held its first meeting on June 8, 1915, under the Chairmanship of Mr. Frederick R. Farrow, F.R.I.B.A., with Mr. H. Kempton Dyson as Honorary Secretary. The publication of the report has been delayed by the war, but copies * are now available and merit the detailed consideration of all structural designers.

Before considering the Report in detail it should be pointed out that the Report cannot alter the provisions of the Act, but by giving a liberal interpretation to it the Report can and does enable part of the objection to certain clauses of the Act to be removed.

If we wish to alter any of the clauses of the Act, and the writer certainly does, we must press for a new Act of Parliament; this Act should not specify the rules of design in detail but should empower the authorities to make regulations. The advantage of this course is that it is possible to amend regulations without setting into operation the heavy machine required for a new Act of Parliament.

FIXITY OF END OF PILLARS AND WORKING STRESSES THEREFOR.

These two questions must be considered in relation to each other. The Act specifies the permissible stresses for three methods of end fixing, viz., both ends fixed; one end hinged and one end fixed; and both ends hinged. The stresses specified for the first case are conservative, those for the second are low, and those for the third are absurdly low. Up to the present most District Surveyors have adopted the attitude that the method of fixing obtaining usually in steel-frame buildings does not conform to the conditions involved in the theoretical consideration of both ends fixed, and the writer believes that this attitude is sound; the difficulty, however, arises that if we take the practical column as equivalent to one end fixed and one end hinged, we are driven to use much lower stresses.

In discussing this matter in detail it is believed best to consider only the range of slenderness ratio which usually occurs in practice; this has been taken as being from 60 to 100, averaging about 80.

Fig. 1 shows the working stresses allowed by the L.C.C. Act for the three methods of end fixing over this range and also those given by the formula favoured by the writer.*

The Report under consideration gives considerable relief in certain cases by allowing both the ends of pillars to be considered as fixed under certain conditions which may be summarised as follows:

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* Published by the Institution of Structural Engineers. Price 1s.

* Discussed in detail in *Concrete and Constructional Engineering*, March, 1918, and June, 1919.

(*a*) The pillar must be continuous throughout its length and, if not rolled in one length, must be jointed near a beam level by a joint complying with specified conditions.

(*b*) A beam must be connected to the pillar in the direction of the radius of gyration under consideration by closely-fitted angle-cleats above and below in addition to any necessary gusset-brackets;

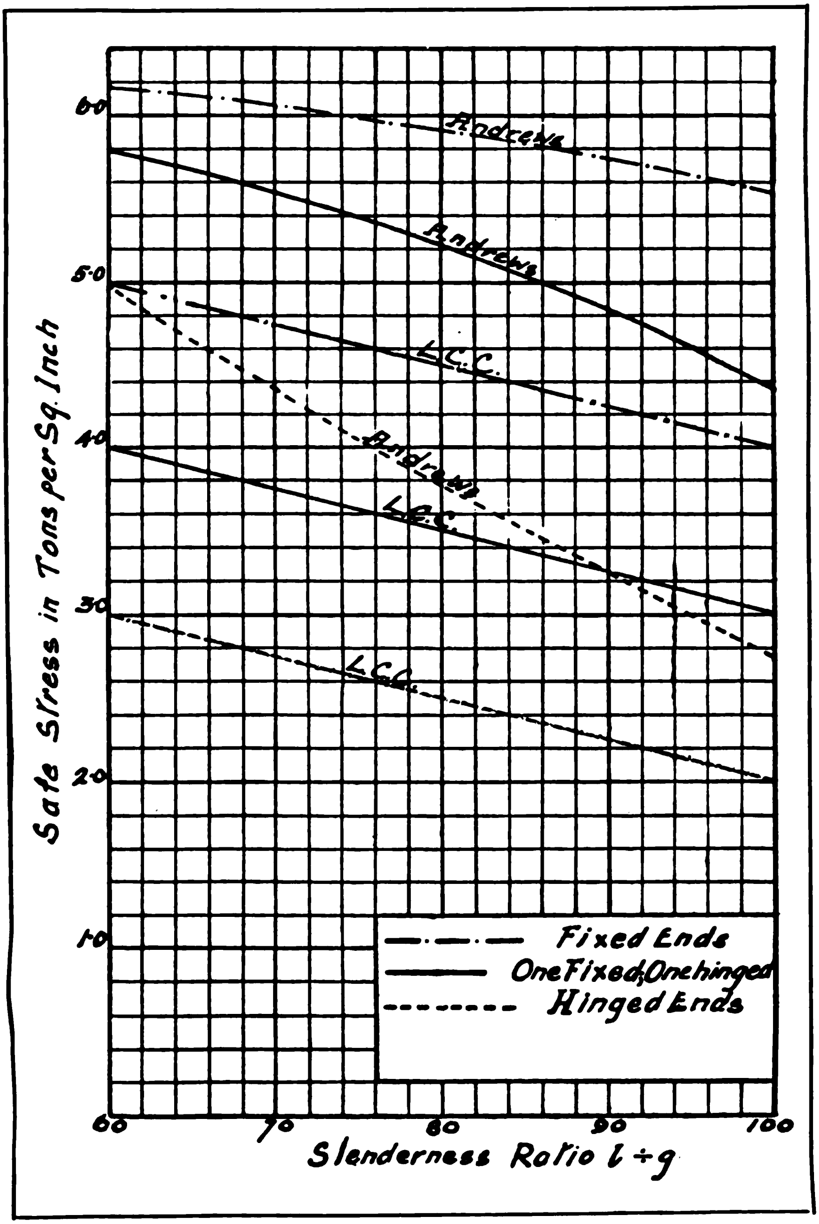


Fig. 1.—Working Stresses for Centrally-loaded Pillars. L.C.C. Figures Compared with Proposed Alternatives.

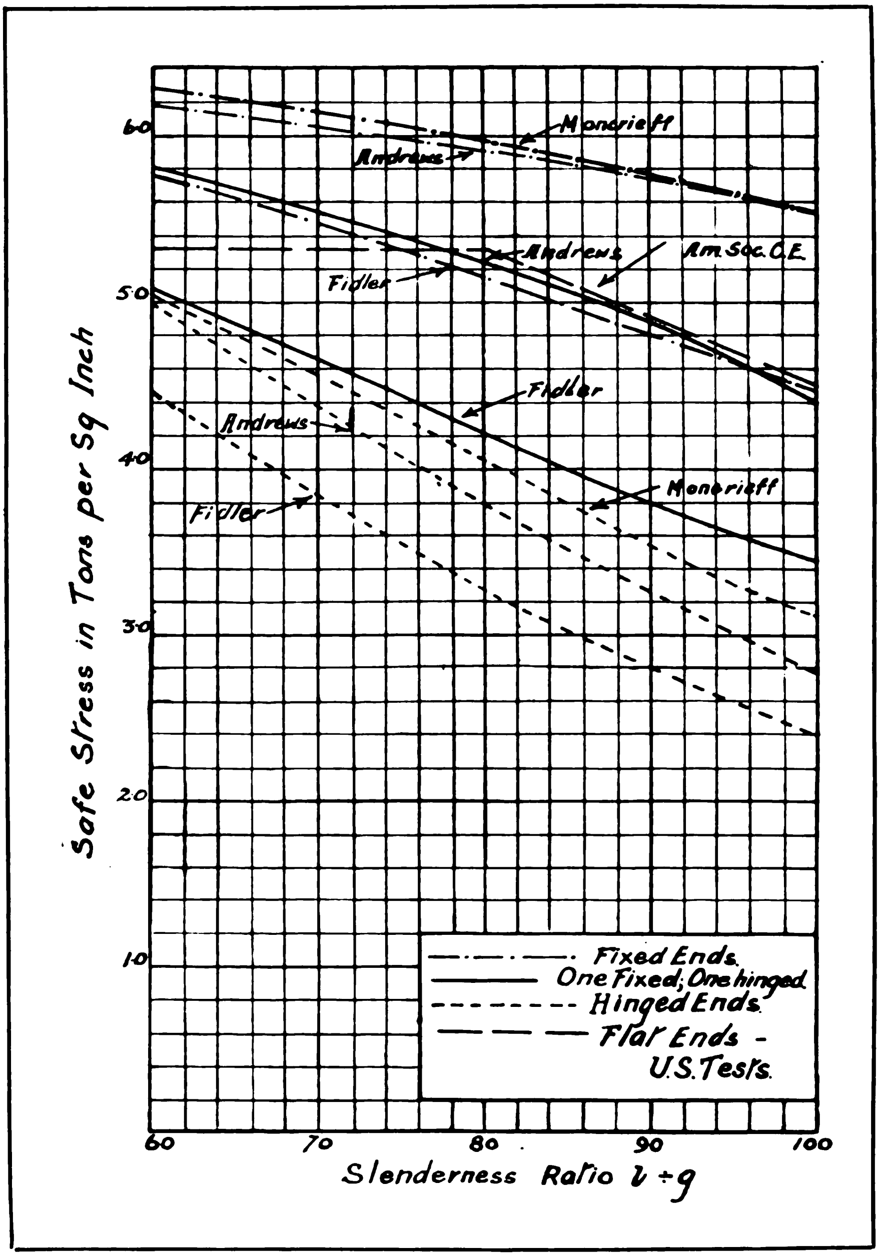


FIG. 2.—WORKING STRESSES FOR CENTRALLY-LOADED PILLARS. VARIOUS FORMULÆ COMPARED.

a formula is given for determining the minimum number of rivets in the cleats.

Referring to *Fig.* 1, it will be seen that for the slenderness ratio of 80, which is about the usual value in practice, the effect of this concession is to allow the working stress to be increased from 3·5 tons per sq. in. (full L.C.C. line) to 4·5 tons per sq. in. (chain-dotted L.C.C. line), amounting to a possible saving in steelwork of something over twenty per cent.

The practice favoured by the writer is to treat the pillar occurring in the usual steel-frame building as equivalent in strength to the condition assumed in the theoretical analysis as "one fixed, one hinged," and for the slenderness ratio 80 considered above this gives a working stress of 5·2 tons per sq. in., which is higher than the L.C.C. value for both ends fixed.

COMPARISON WITH OTHER PILLAR FORMULÆ.

In order to compare the figures favoured by the writer with those favoured by other designers, and to save confusion by showing too many curves on the same diaphragm, on *Fig.* 2 are also shown the figures according to Moncrieff, Fidler, and the recent report of the American Society of Civil Engineers. Fidler's figures are lower than the others but are higher than the L.C.C. figures, and are based upon the well-known and very thorough investigation of the subject many years ago by Mr. Claxton Fidler. Moncrieff's figures were obtained from a theoretical investigation agreeing well in general assumptions but differing in mathematical detail from that of the present writer (see exhaustive paper on the subject in Vol. XLV of the *Proceedings* of the American Society of Civil Engineers); he does not take as a standard method of end-fixing "one end hinged, one fixed," but takes a case of "flat ends," the safe stresses for which are the same as for both ends fixed "up to a value of the slenderness ratio of 106," which exceeds the limits of the range of our present discussion. The U.S. figures are important and are given in a lengthy report in the *Proceedings* of the American Society of Civil Engineers for December, 1917. This report was made by a special committee appointed to investigate the problem and to carry out special tests; the tests were made on pillars with flat ends and the figures recommended do not provide for other methods of end fixing, it being apparently assumed that the flat-ended pillars approximate as nearly as possible to the pillar as usually employed in buildings. One interesting and important point in regard to these U.S. figures is that the report points out that the figures recommended are lower for small slenderness ratios than those usually adopted in American practice. It will be noted that these figures are in striking agreement with those given by the writer's full-line curve based upon mathematical treatment of the problem.

ECCENTRIC LOADING OF PILLARS.

In accordance with the L.C.C. Act the bending stresses due to eccentricity of loading of pillars must be calculated and added to the direct stresses; the combined stress must not exceed the tabulated stress for the given slenderness ratio and method of end-fixing. The Act does not specify how this calculation of bending stress is to be made, and in the past it has been the usual practice to calculate the stress as if the full bending moment due to the eccentricity has to be carried by the section below the load.

The Conference Report under consideration proposes to give some relief in this respect by allowing the bending moment, in all lengths of a continuous pillar except the top length, to be taken as *one-half* the eccentric load multiplied by the eccentricity. It also makes clear the much-discussed problem as to what is to be taken as the eccentricity; the recommendations in this respect are that the eccentricities shall be taken as follows:—

Single web connections.—One-half the projection of the cleat or 2 in., whichever is greater.

Double web and flange connections.—One-half the thickness or diameter of the pillar measured in the direction of the length of the beam.

This proposal does give us welcome relief in some cases, but does not deal with the writer's fundamental objection to the procedure of making the combined stresses equal to the tabular stresses; this, of course, is not the fault of the Conference since it cannot depart from the specific provisions of the Act.

SUGGESTED MODIFICATION OF TREATMENT.

This question of eccentric loading of pillars is one which has been a bone of contention for years, and is complicated by the fact that full theoretical treatment of the subject lands us into deep mathematical water. It is, however, of vital importance because in practice almost every pillar is eccentrically loaded.

The method of treatment favoured by the writer will be explained with reference to *Fig.* 3, which is based upon theoretical considerations explained in previous articles already referred to. Various other theoretical formulæ, such as that known as the " Modified Euler " or " Eulerian secant," and the approxi-

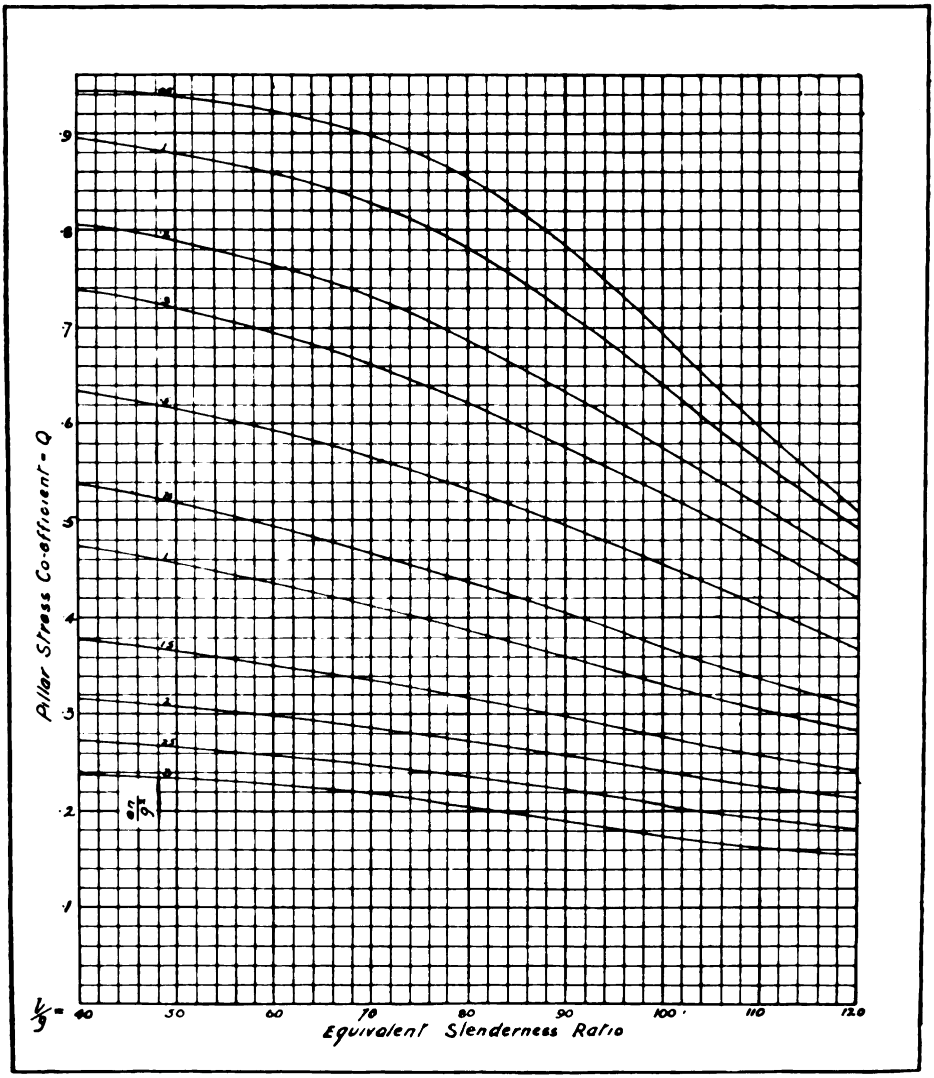


Fig. 3.

mation to the last-named known as Perry's formula, have been adopted and all give similar results.

The diagram contains a number of curves drawn for the pillar stress coefficient plotted against the slenderness ratio for various values of the quantity $\frac{en}{g^2}$ where

e = eccentricity of load
or $\left(\frac{\text{bending moment}}{\text{load}}\right)$
n = distance from centroid of section to extreme compression edge.
g = radius of gyration of section.

In determining the safe eccentric load to be carried by any given pillar we proceed as follows:

(*a*) First find $\frac{en}{g^2}$.

(*b*) Find the equivalent slenderness ratio or buckling factor taking into account the method of end fixing; to do this take $0{\cdot}5l$ for fixed ends, $0{\cdot}7l$ for one end hinged and one fixed, l for both ends hinged, and $2l$ for one end fixed and the other end free. Treat the continuous pillar as equivalent length $0{\cdot}7l$.

(*c*) Read up from the diagram at the bottom scale to the approximate curve, interpolating if necessary, and run out horizontally to the Q scale. Multiply the basis stress for the steel (6·5 tons per sq. in. in the usual case) by Q, giving the safe eccentric stress c_E; the safe eccentric load $= c_E{\cdot}A$, where A is the area of the section.

If, for instance, $\frac{l}{g} = 60$ and $\frac{en}{g^2} = 1{\cdot}8$ we should regard the value of Q as 0·32, so that the safe eccentric stress per sq. in. becomes $0{\cdot}32 \times 6{\cdot}5 = 2{\cdot}08$ tons per sq. in.

In cases in which the eccentricity is small the result of calculating in this manner will not differ very seriously from the results obtained by the method specified by the Act, except for the point already noted as to the low stresses allowed for central loads, but for cases in which the eccentricity is appreciable, especially in single story workshop buildings in which the pillars have to carry eccentric crane loads, the suggested method results in considerable saving in material.

OBLIQUE LOADING OF PILLARS.

By oblique loading of pillars is meant eccentric loading not on one of the principal axes only. In this case the Report makes clear the point, which has been neglected by many designers, that the bending stresses in respect of both of the principal axes should be calculated and added together in order to obtain the total bending stress.

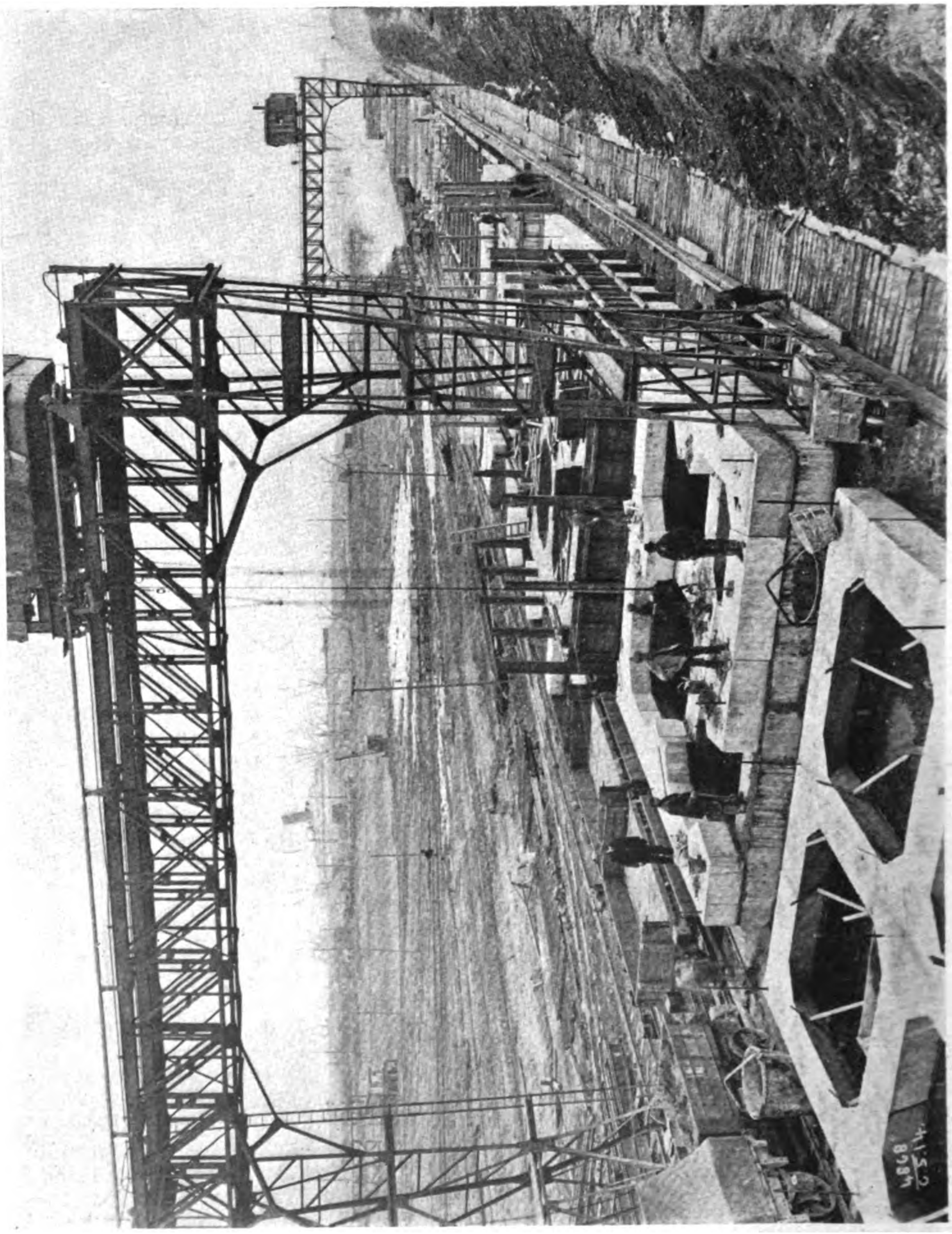
GENERAL NOTE ON THE CONFERENCE REPORT.

A number of other interesting points are dealt with in the Report which consideration of space prevents us from treating fully in the present article; we will deal briefly with some of the principal ones which are likely to affect designers' practice.

On the question of impact allowances for live loads, the Report recommends that the equivalent dead load for lifts, etc., shall be taken as three times the weight of the cage and load; for rolling loads 50 per cent. increase; and for other loads that might be suddenly applied to a floor, without impact, 100 per cent. increase.

The subject of riveting is dealt with in detail, and it is recommended that the diameter of the hole shall be the diameter of the rivet calculated.

The Report is accompanied by four appendices. The first deals with the stresses in mansard roofs, and is contributed by Mr. H. Kempton Dyson. The treatment is based upon Castigliano's Theorem, and the resulting formulæ, which have a rather terrifying appearance, are a variation of the formulæ resulting from the analysis of the two-hinged rigid arch; Mr. Dyson is to be congratulated on a very good piece of work, which is a real addition to the literature of the subject. The second Appendix is by Mr. W. G. Perkins and deals with dimensions of rivet heads. Mr. Dyson contributes a note in Appendix III upon shear on rivets in compound girders, and the last Appendix is written by Mr. W. G. Perkins and Mr. E. Fiander Etchells on the design of splice plates.

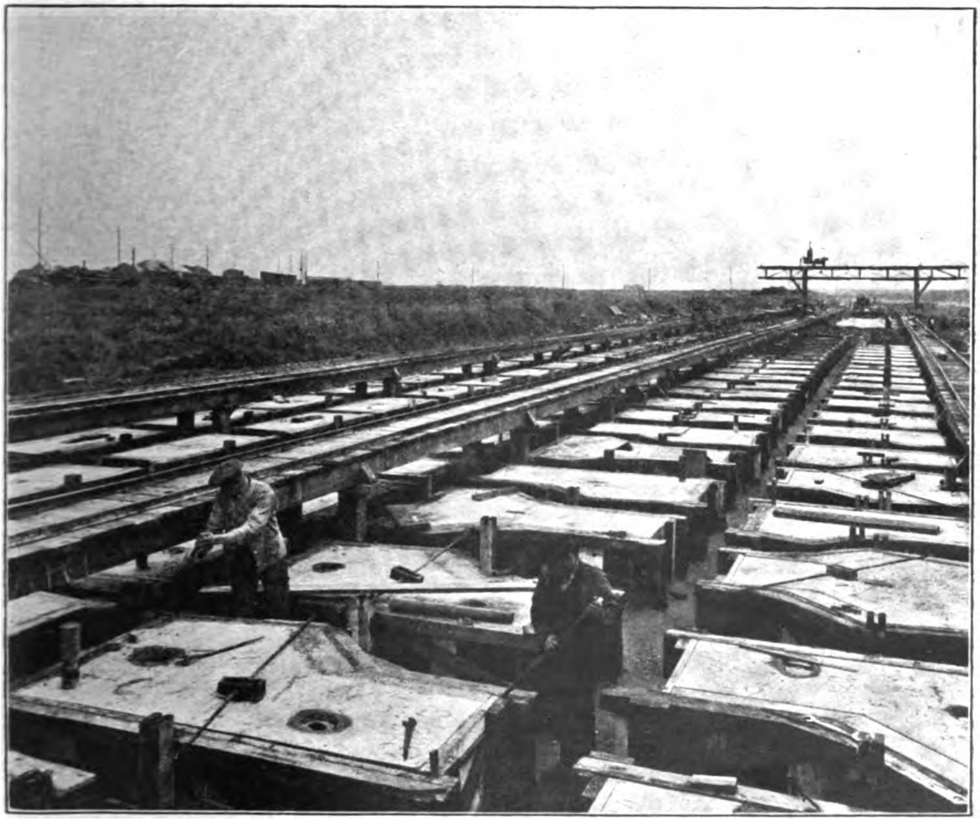


Tilbury Docks Extension: General View. (*See p.* 147.)

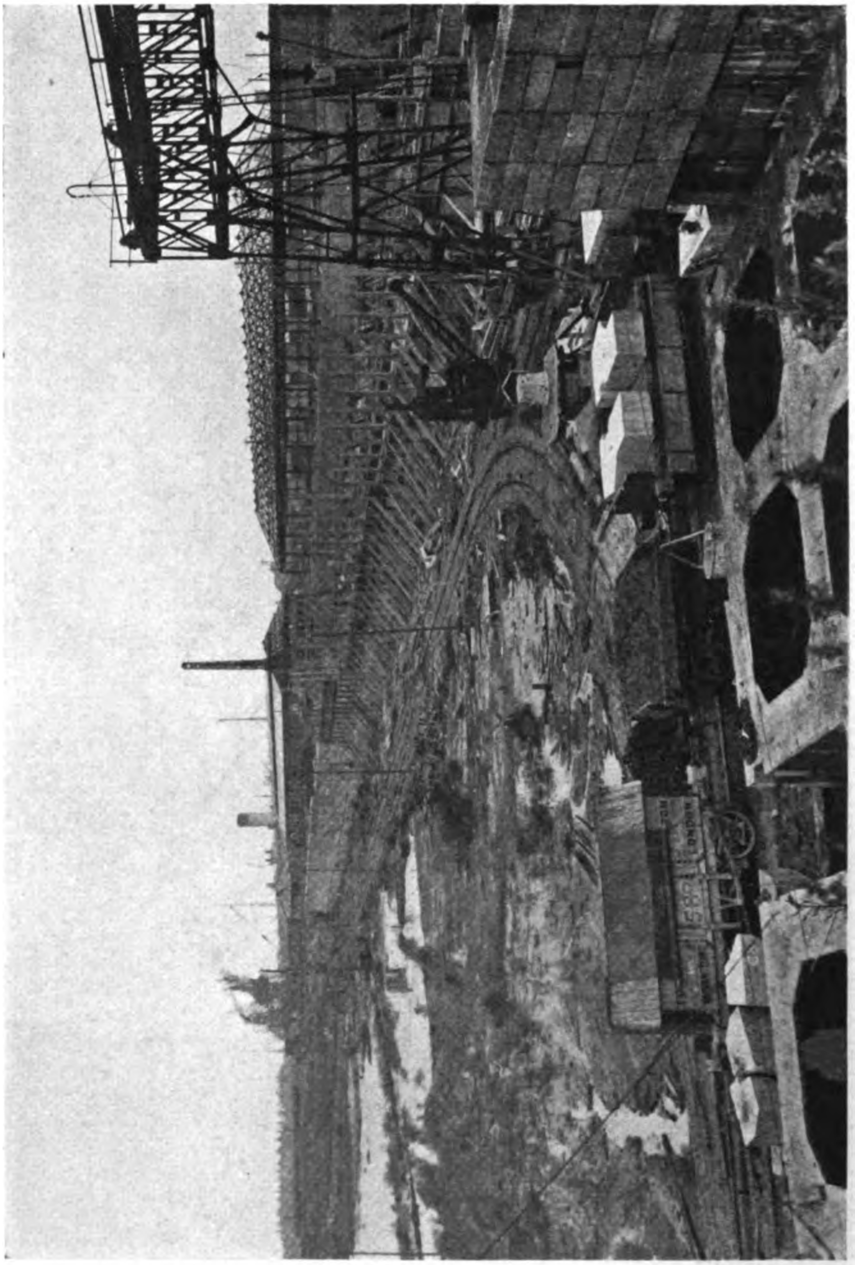
EXTENSIONS AT TILBURY DOCKS.

In the course of a paper, read recently before the Institution of Civil Engineers, Mr. F. M. G. Du-Plat-Taylor, M.Inst.C.E., described the extensions at Tilbury Docks carried out between 1912 and 1917.

The works described consisted of the extension of the main dock for a distance of 1,611 ft. westwards with two new masonry quay walls, 1,611 ft. and 361 ft. long respectively on the south and west sides, but with an earthen embankment, faced with chalk, on the north side, so as to admit of future extension in that direction. The depth of water provided for alongside the greater part of the new quays was 42 ft. 6 in. The subsoil over the whole site consisted of alternate layers of clay and peat down to the ballast, which afforded the only stable foundation, occurring at depths ranging from 38 ft. to 45 ft. below O.D. The level of the surface, which was intersected by dykes and drains, was about 9 ft. above O.D. The clay which formed the bulk of the excavation weighed about 90 lb. per cu. ft. in its natural state and, when dried, shrank to about two-thirds of its normal bulk. Its angle of repose was vertical when dry and about 26 deg. 40 minutes when wet. Loading tests at ground level showed that it would not carry more than 12 lb. per sq. ft. without some subsidence, and that the subsidence increased directly with the load up to 8 cwt. per sq. ft., when it reached 2·45 in. With higher loading it increased more rapidly, so that at 13½ cwt. it was 17 in. Apart from this low bearing capacity there is a regular subsidence of the



Moulding Yard of Concrete Blocks—Tilbury Docks Extension.

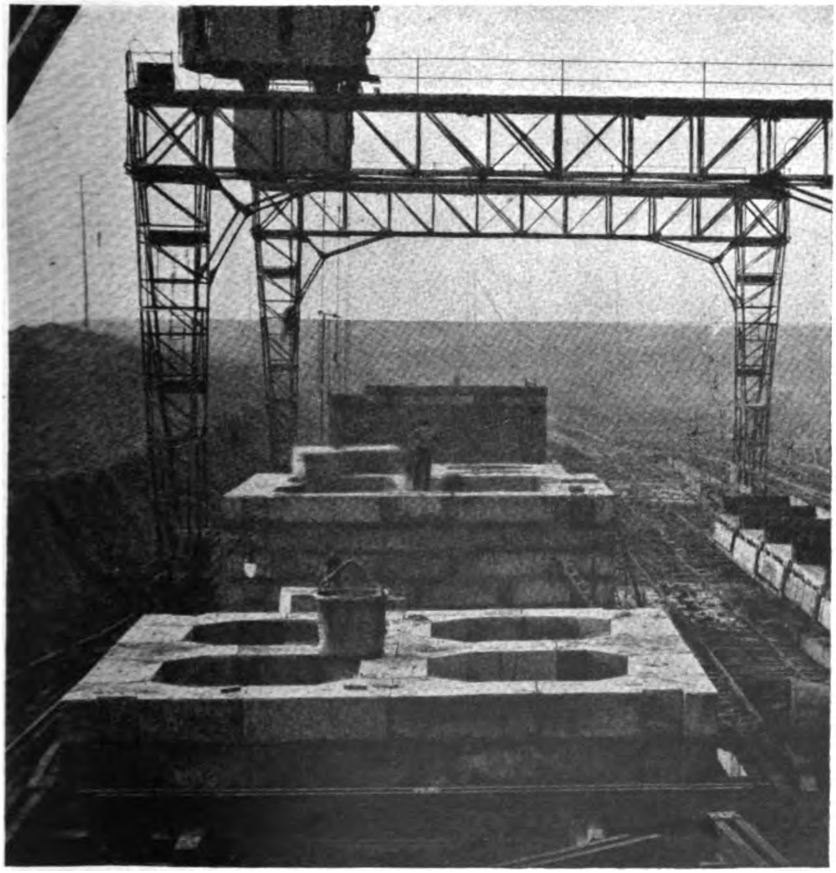


Tilbury Docks Extension: Showing Construction of Wall Superstructure and Quay Sheds.

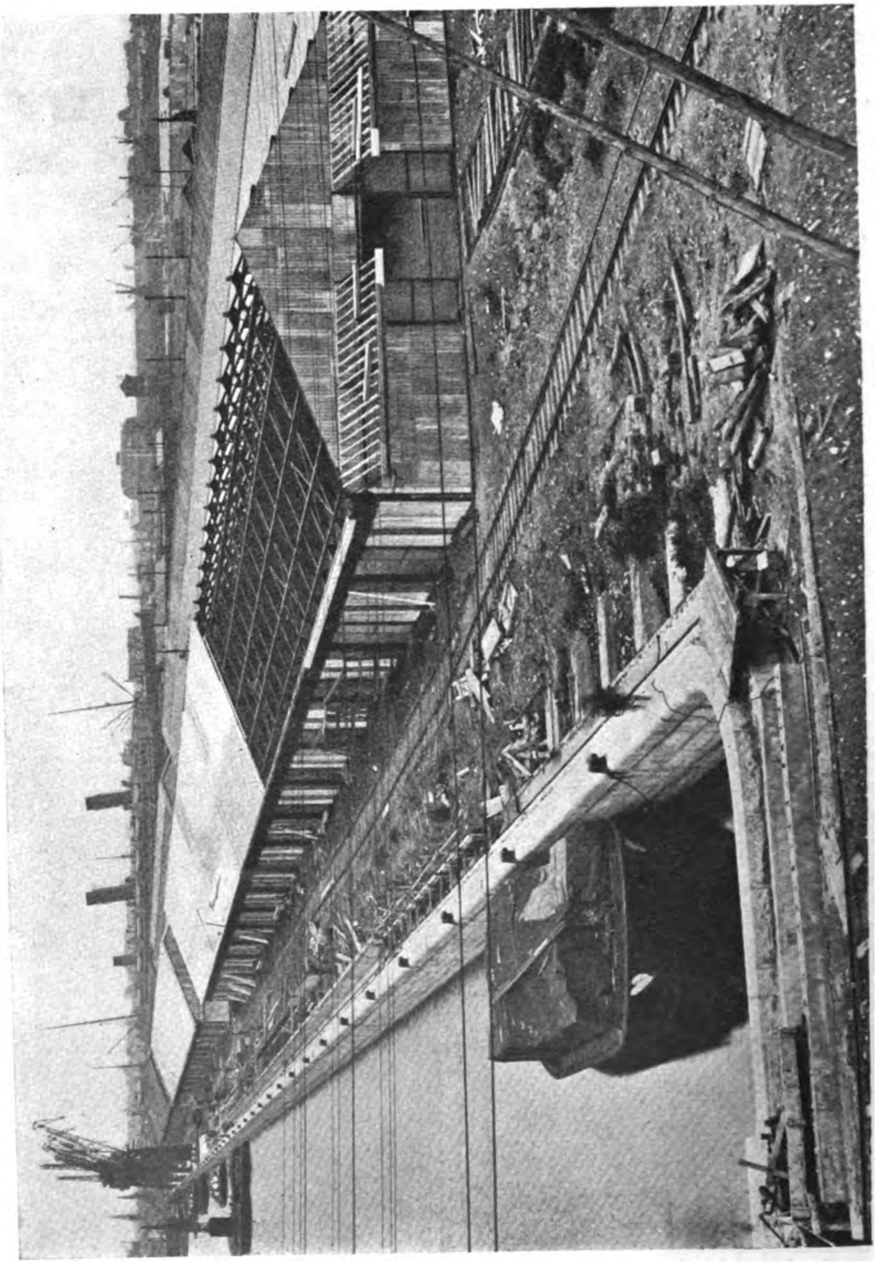
ground due to the gradual alteration and shrinkage of the peat strata.

In the first instance several trenches were opened; but trouble was experienced from the ground rising in the bottom and from the lower ends of the runners coming inwards, and eventually construction in trenches was abandoned. It was then decided to carry out the quay wall in monolith construction, with the exception of a short length at the east end of the south wall, adjoining the old quay wall, which was constructed in a trench as a mass concrete wall founded at 29·5 ft. below O.D. and carried on 139 vertical and 14 inclined reinforced concrete piles driven into the ballast.

The monoliths were constructed of precast blocks. They were pitched with a space of 6 ft. 5 in. between adjoining monoliths, the number of monoliths required for the south quay wall being 43, and for the west wall 10. The monolith blocks were 6 to 1 concrete, with radial joints adapted to resist external pressure. The shoes were made of steel joist sections and had no cutting edge. Sinking was carried out by grabbing within the wells, and was assisted as necessary by the use of cast-iron kentledge blocks, of which the maximum weight placed on any monolith at any one time was 1,000 tons. The spaces between monoliths were closed back and front by driving timber sheet piling, and after the monoliths had been sunk to the full depth, *i.e.*, about 2 ft. into the ballast, these spaces were, together with the monolith wells, cleaned out and sealed with 5 to 1 concrete deposited under water. When the mono-



Sinking Well Foundations—Tilbury Docks Extension.



TILBURY DOCKS EXTENSION: WORKS NEARING COMPLETION.

liths in the course of sinking reached a critical depth, which varied with their position in relation to the old dock and river, water came into the wells and henceforth rose and fell with a lag behind the tides and with less amplitude than the tidal rise and fall.

To prevent any pressure being put upon the sealing concrete before it set, a steel vent-tube was inserted in each seal and finally closed by grouting after the seal was hard. The wells and spaces were then pumped out and filled with concrete, except the front wells, which were left empty to reduce the pressure on the foundation beneath the toe.

In moulding the blocks, the sand and stone in the ballast were washed and separated, and then re-mixed in the mixers in predetermined proportions according to the ascertained voids in the stone. Experiments showed that the best admixture was 5 per cent. more sand than the voids in the stone, and that there was a more rapid deterioration in strength with an excess of sand beyond 5 per cent. than there was with a like deficiency of sand.

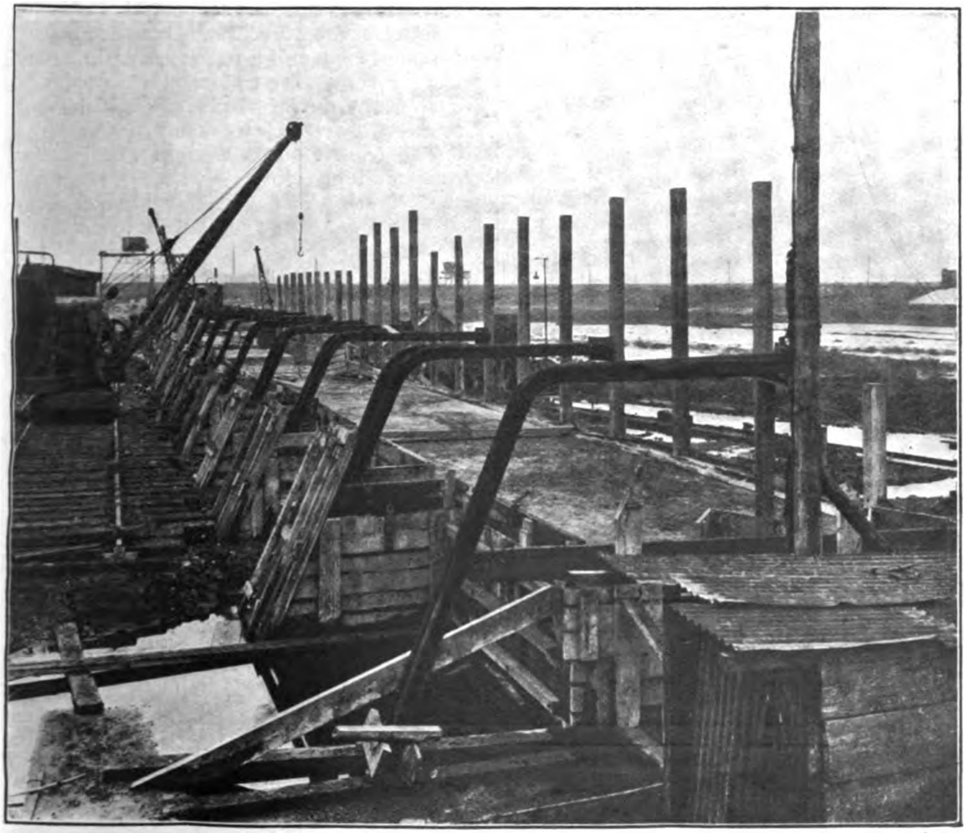
Records were kept of the resistance to sinking of the monoliths, and of bearing resistance.

(1) From observation of the initial subsidence before any grabbing was begun, the resistance to bearing was 80 lb. to 100 lb. per sq. ft. in dry material, falling to 12 lb. to 15 lb. in wet.

(2) Skin friction in more or less dry clay and peat averaged 877 lb. per sq. ft.

(3) Skin friction in clay and peat with water rising to the top of the monolith averaged 855 lb. per sq. ft.

(4) Skin friction after the monolith had entered the ballast averaged 873 lb. per sq. ft.



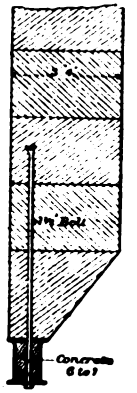
Superstructure of Wall: Showing Rail Reinforcement—Tilbury Docks Extension.

The friction recorded was the friction of rest. From observation, the action was to shear off a layer of clay, etc., which sank with the monolith, and the friction recorded was that between surfaces of clay, etc., and not between concrete and clay.

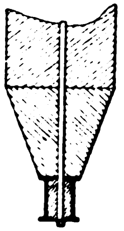
Whatever bearing resistance existed is included in these figures, it being impossible to separate this from the frictional resistance; but as the wells were grabbed well below the bottom of the shoes before any movement took place, the bearing resistance can only have been small. The foregoing figures are very high in comparison with previous records, which is attributed to the nature of the peat layers, as it was observed that great difficulty was experienced in drawing piles driven through the peat, as compared with the effort required to draw piles driven in clay and similar strata. The rate of sinking the monoliths averaged 21·57 ft. per week.

A superstructure wall was built upon the monoliths, in which reinforced counterforts were provided, and which carried crane rails, supported between counterforts by a reinforced concrete continuous beam, and a subway for pipes and electric-conductor conduit for supplying current by means of plough collectors to twelve 3-ton electric travelling quay cranes.

The transit sheds were of steel-frame construction, clothed with corrugated sheet-steel, and covered by Italian steel or zinc roofing, the columns being carried upon reinforced concrete piling. The floors were partly of granolithic concrete paving and partly of wood.



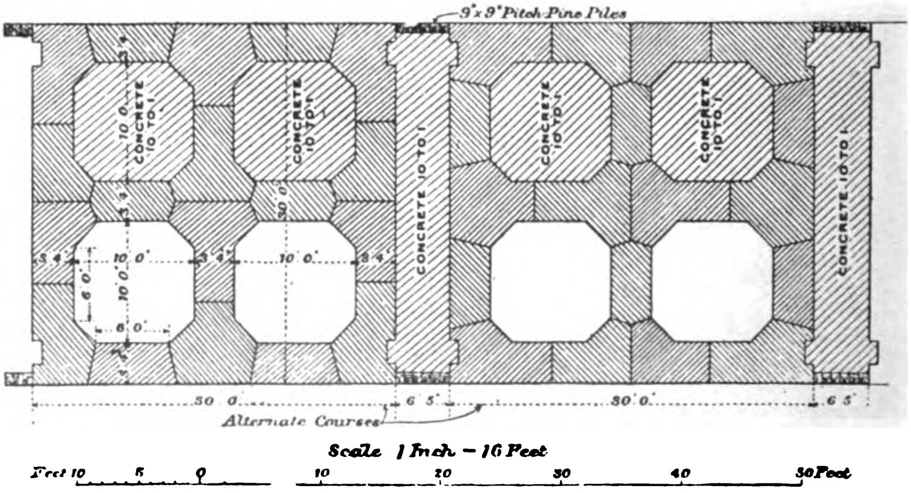
DETAIL OF LOWER OUTSIDE BLOCKS & SHOE



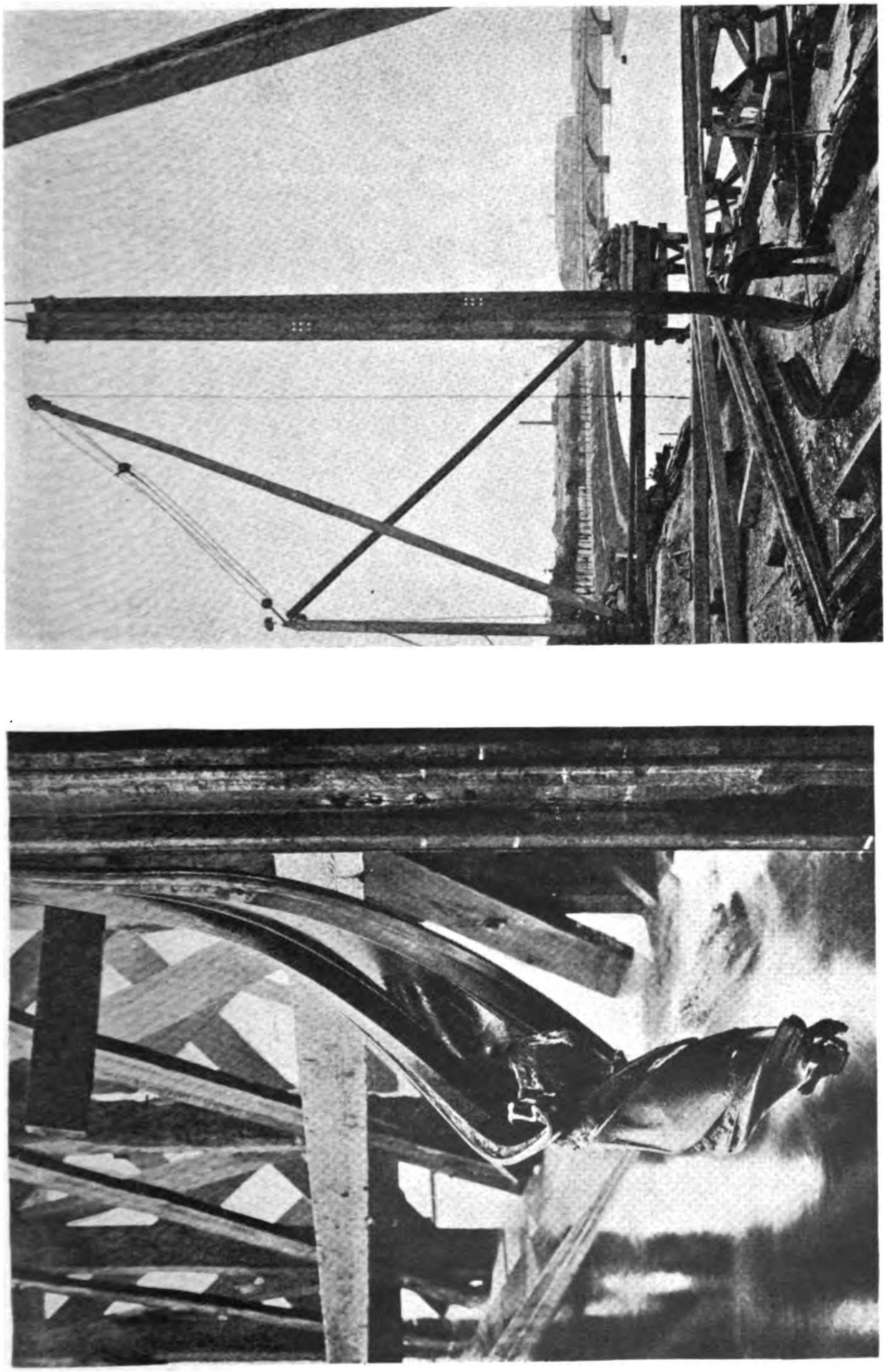
DETAIL OF LOWER CENTRE BLOCKS & SHOE

Scale 1 Inch – 6 Feet

Feet 1 ½ 0 1 2 3 4 5



HORIZONTAL SECTION OF MONOLITHS.



Figs. 4 and 5.—Damaged Piles.

REINFORCED CONCRETE PUMPING-STATION AT ABERDEEN. (*See p.* 155.)

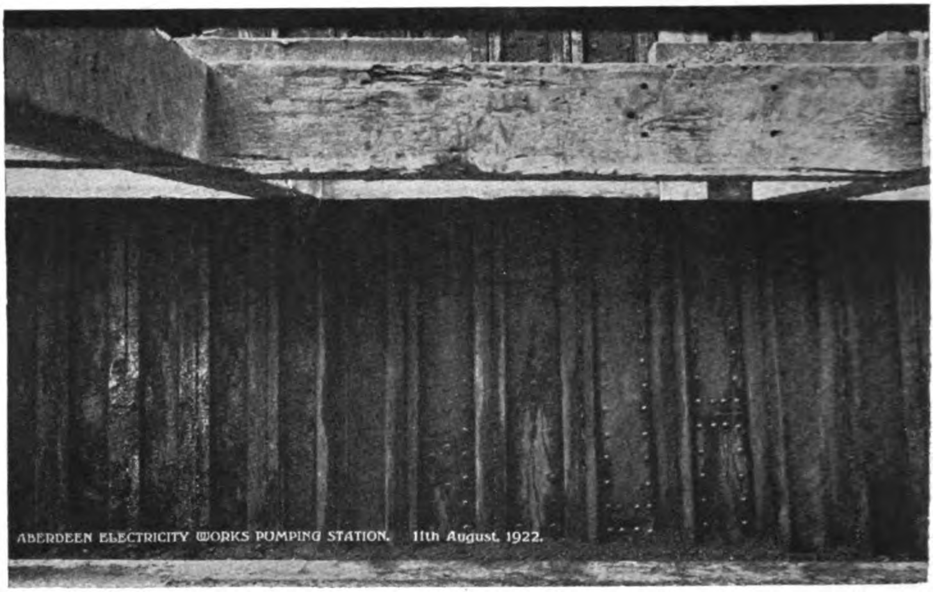

ABERDEEN ELECTRICITY WORKS PUMPING STATION. 11th August 1922.

FIG. 6.—TAPERED PILES.

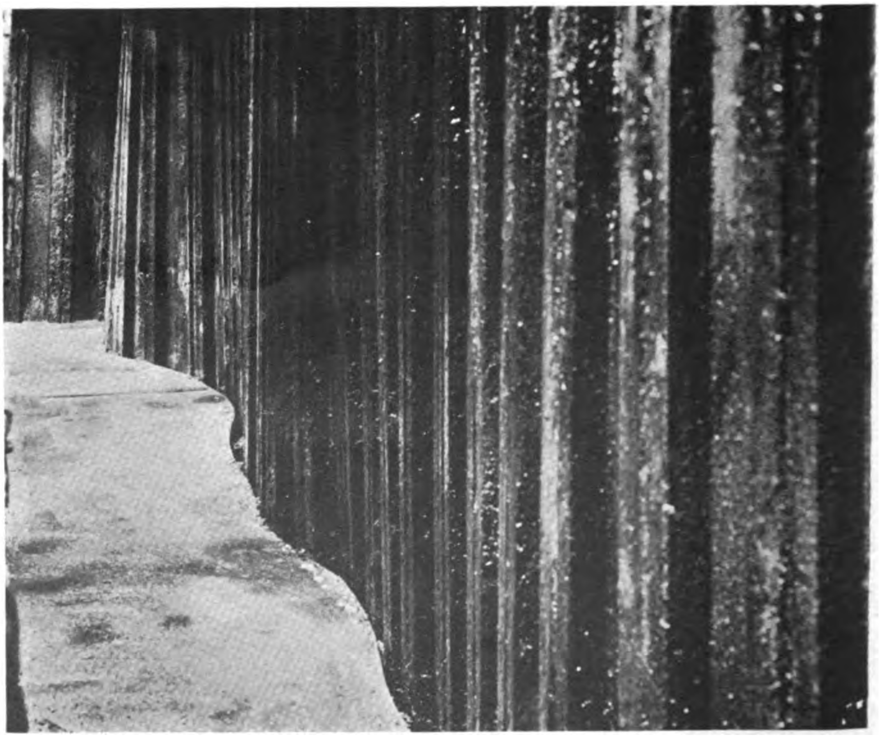


FIG. 7.—DISTORTED PILING.
REINFORCED CONCRETE PUMPING-STATION AT ABERDEEN. (*See p.* 155.)

REINFORCED CONCRETE PUMPING-STATION AT ABERDEEN.

By JAMES MITCHELL, M.Inst.C.E., M.I.Mech.E.

THE generating station of the Aberdeen Corporation electricity undertaking is situated about 300 yards from the River Dee. The condensing water for the station is obtained from the river by means of electrically-driven centrifugal pumps contained in an underground chamber built on its north bank, and having suction-pipes projecting into the river sufficiently far to be well covered at low water. Owing to the development of the generating station the pumping capacity has become barely sufficient, especially in view of the difficulty experienced in keeping the perforated ends of the suction pipes free from floating matter, such as leaves, grass, twigs, and river and sea weeds which have to be removed at frequent intervals by means of long-handled wire-brushes operated from a boat at low tide. This is a very unsatisfactory arrangement, because it not infrequently happens that while the tide is sufficiently low the current of the river is too strong to permit a boat to be used. In winter there is sometimes the added difficulty of floating masses of ice.

The new pumping station (*Figs.* 1, 2 and 3, pp. 156–7) now nearly completed, consists of a pump chamber, a sump chamber, and an inlet culvert, all constructed of reinforced concrete. The pump chamber and sump chamber are entirely underground, and the floor-level of the inlet culvert is 3 ft. below the lowest recorded water-level of the river.

The sump chamber is divided into two equal portions by a wall which extends above the highest recorded river level, and a continuation of this wall extends the whole length of the inlet culvert, dividing it also into two equal portions, each portion having at its junction with the sump chamber a vertical sluice with an opening of 8 ft. by 6 ft. and operated by a motor, hand-gear being provided for emergencies. The sump chamber thus consists of two entirely independent units.

The sump chamber is circular in plan up to about mean-tide level, above which it is square. In it there are two centrifugal pumps, having delivery-pipes 22 in. in diameter, and each driven by a 400 B.H.P. electric motor. As the pumps are intended to be controlled by switches at the generating station it is important that they should be placed well below the level of the lowest tide, so as to avoid danger from loss of priming. They are, therefore, of the vertical-spindle type, the motors being carried at about mean-tide level on steel girders having ample stiffness to minimise vibration. The suction pipes are 42 in. in diameter, and are arranged so that either one or both of the pumps may draw from either one or both of the units of the sump chamber. The outer end of the inlet culvert is covered by a screen of perforated galvanised steel-plate. In addition to this, provision has been made for the installation of revolving screens in the sump chamber should they prove to be necessary. There are grooves in the side walls of the culvert at its outer end, in which temporary timber bulkheads may be fixed so that either one of the units of the sump chamber may be pumped empty for the purpose of removing mud or overhauling the sluice or the revolving screens.

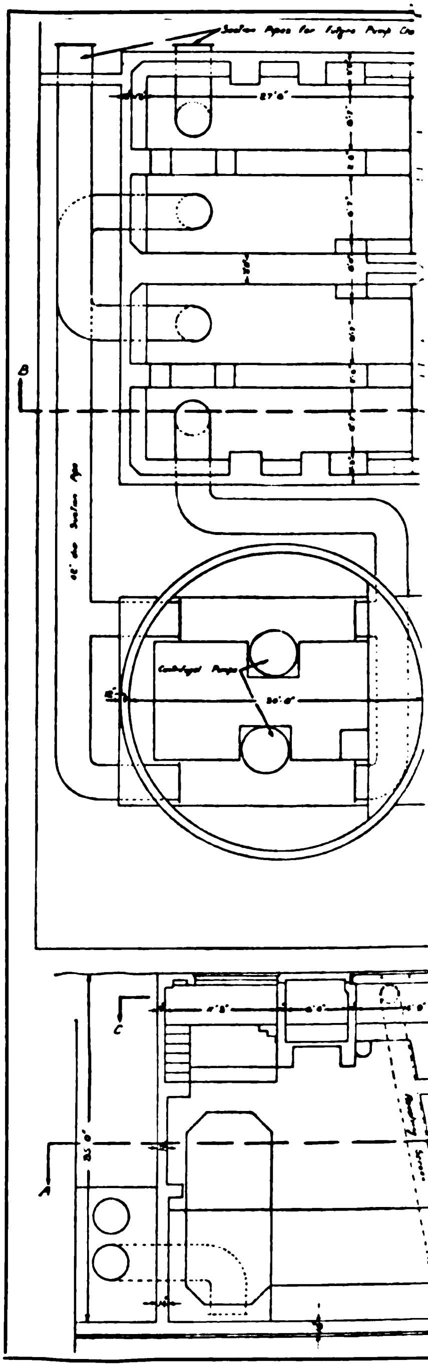
The pumping capacity of the new station will be about 47,000 gallons per hour. It is intended, when justified by the expansion of the generating-station, to build on the down-river side of the sump chamber another pump chamber precisely similar to the new one, so that the one sump chamber will serve both sets of pumps. The suction-pipes for the proposed pump chamber have been incorporated in the present work, and provision has been made for facilitating the construction of the chamber when it becomes necessary.

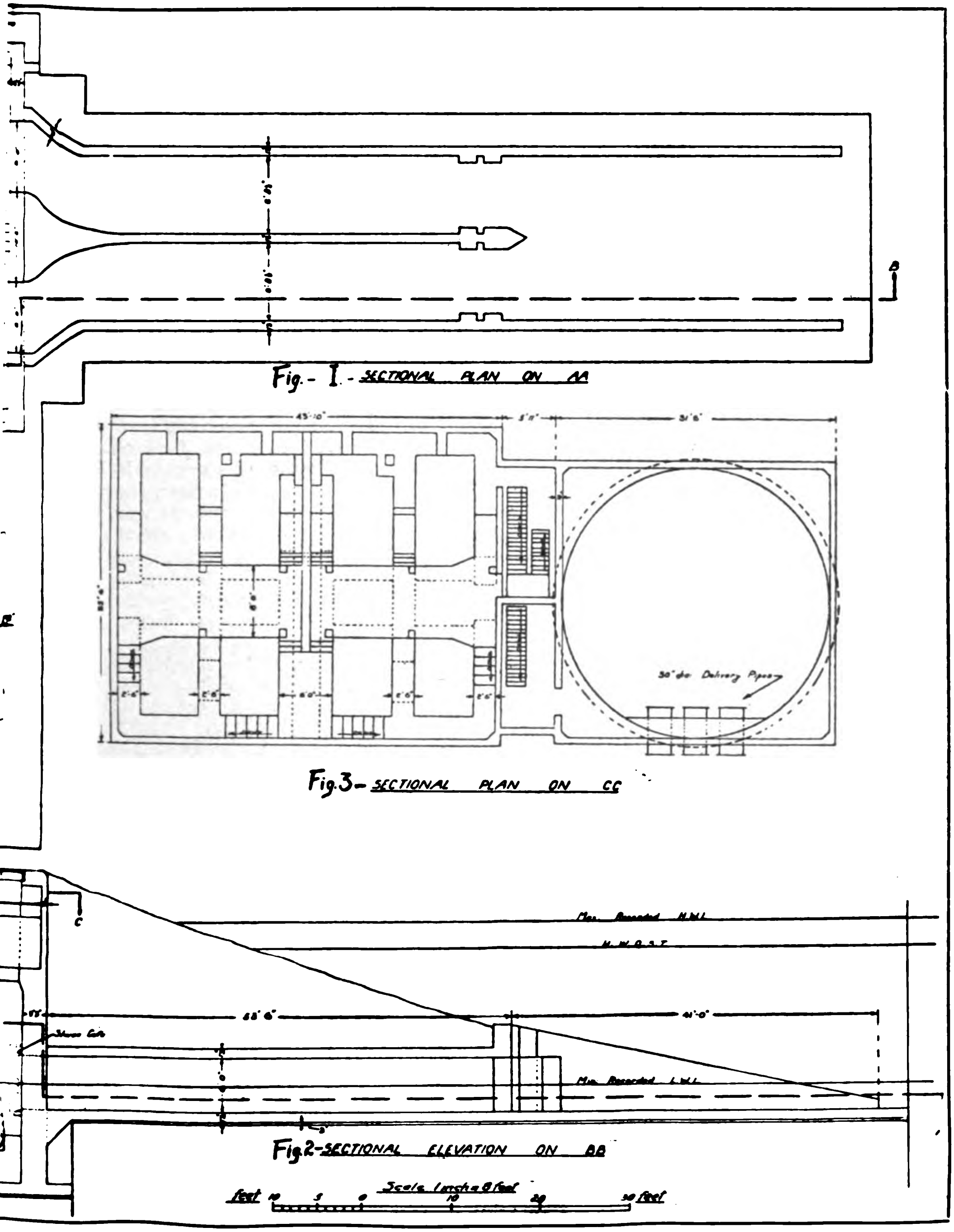
Owing to the proximity of a busy street, under which one of the main sewers of the city runs, it was specified that the whole site of the works had to be enclosed by steel interlocking sheet-piling, driven to the lines shown on the drawings, but the lengths of the piles and the design of the cofferdam were left to the decision of the contractor, subject to the approval of the engineer.

It was finally arranged that on the landward side of the river-bank, the top of the piling should be 4 ft. above the level of high-water of ordinary spring-tides, and that on the riverward side, in order to allow for wave motion and for floating ice, the top-level should be 1½ ft. higher. Those levels proved to be none too high, as on December 17, 1921, high-water level in the afternoon was fully 3½ ft. above H.W.O.S.T. and 7½ in. higher than the previous highest record. In order to provide against the erosion of the river-bed, which was expected to occur owing to the projection of the river-arm of the cofferdam into the current of the river, the piles of that part were driven about 16 ft. into the ground. In the landward portion of the cofferdam the piles extended about 8 ft. below the foundation-level. A sump, having a depth of about 4½ ft. below foundation-level, was provided by extending the up-river end of the main trench about 6½ ft. beyond the limit of the concrete structure.

The work of handling the steel piles, the excavations, and the bracing-timbers of the cofferdam, the placing of the concrete in the upper portions of the pump and suction chambers, and the withdrawing of the steel piling, was chiefly done by a 7-ton steam derrick crane.

The steel piles were driven by means of a piling machine 55 ft. high, equipped with a steam winch and with a steam piling-hammer weighing 53 cwt. A considerable amount of diving work was necessary in clearing away stones in the bed of the river, and about a dozen of the piles had to be withdrawn after driving owing to bending and twisting caused by obstacles, such as boulders and tree trunks. One of the damaged piles is shown in *Figs.* 4 and 5 (p. 153). When the second pile in *Fig.* 5 was withdrawn it was found to be broken almost through at one place, and when lifted by the crane to be photographed the lower portion broke off, and is seen lying on the ground. Some trouble was caused by the fact that as the piling progressed the piles tended to lean more and more forward at the head till at one stage of the work the piling had an inclination of 1 in 10 from the vertical. This was due to the fact that although the line of piling was straight at the top, it was, owing to deflections caused by stones, etc., very irregular at the bottom at some places. To bring the piling back to the vertical,





REINFORCED CONCRETE PUMPING-STATION AT ABERDEEN.

tapered piles were used as shown in *Fig.* 6 (p. 154). A tapered pile was made by cutting an ordinary pile in two longitudinally by an oxy-acetylene flame, separating the two halves by 8 in. at one end, tapering to nothing at the other end, and then riveting two tapered coverplates to them. A steel cap was used in driving the piles. The first caps used were of steel plates and angles riveted together, but so much trouble was caused by fractured rivets that the riveted caps were abandoned in favour of solid steel-castings, which proved quite satisfactory. The piles were all driven from 3 to 6 ft. into a bed of hard boulder-clay, but in spite of this a pile while being driven would on occasion take such a hold of the adjacent one—probably owing to one or both of them having been deflected or burred by stones—as to pull it down with it, in two or three cases as much as 3 ft. In a few cases two piles at a time were thus pulled down after they had been driven to the finished level. In spite of the considerable amount of distortion of the piling revealed during the progress of the excavation, only in three cases was it found that the interlock of the piles had been burst, and fortunately none of them was serious. A subsidence of the roadway due to the largest of those openings was treated by filling it with alternate layers of stable litter and puddled clay. *Fig.* 7 (p. 154) is a photograph of a portion of the distorted piling taken just after the completion of the concrete floor of the suction chamber, and is a tribute to the excellence of the piles.

For the purpose of providing a sluice in the cofferdam, a rectangular hole of the required size was cut in a pile by an oxy-acetylene flame, two heavy channel-bars were bolted over the opening as a temporary strengthening, and the pile was driven to the proper depth. The channel-bars were then removed and a timber sluice fitted, the joint between it and the pile being made watertight by means of oakum and hot bitumen. When it is required to withdraw the sluice pile, the timber sluice will be taken off and the channel strengthening-bars replaced.

For the driving of the sheet-piling of the river arm a timber staging, carrying a railway for the 55-ft. piling machine, was built round three sides of its site. This staging consisted of round Scots-fir piles carrying hardwood crown-beams and braced by 11 in. by 3 in. planks. It was constructed by means of a travelling cantilever piling-machine which was constructed on the works from stock materials. The machine included a 25 ft. piling-frame, a 25-cwt. steam piling-hammer, and a 30-cwt. hand derrick-crane.

The driving of the steel piling was continued round the work from the pile first driven, a junction being effected by continuing the piling for two piles beyond the first-driven one. This junction was very satisfactory, being nearly water-tight. The sluice was kept open, and the tide allowed to rise and fall within the river arm of the cofferdam until the piling was completed. When the sluice was closed and pumping began there was a great deal of leakage through the joints of the piling in the river arm. By the use, however, of a few tons of ashes, dropped into the water outside the piling in small quantities at a time, the dam, at the end of about a fortnight, was rendered remarkably watertight, about 90 per cent. of the water which was pumped thereafter coming from the springs met with in the ground.

The materials passed through during the process of excavation were typically estuarine in character, and consisted of first about 25 ft. of clean sandy-gravel, then about 7 ft. of silty sand, and finally—in the main trench—about 5 ft. of a mixture of peat, gravel, and clayey mud. The ground contained a considerable number of boulders and large tree-trunks. When foundation level was reached, a great number of springs were found.

The inlet culvert is founded on a bed of very fine sand, through which at foundation level arose a number of springs. Sheet piling was therefore driven across the inner end of the river arm, and the whole site of the culvert covered with a layer of gravel, so as to allow a free passage for the water, while preventing the sand from being carried with it into the deeper portions of the trench and from thence into the sump.

When the excavation had reached a depth of about 18 ft., a 6-in. centrifugal pump and a 4-in. pulsometer were installed at the outer end of the river arm, as this formed a ready-made sump. When a depth of about 27 ft. had been reached the main sump was formed, and

6-in. pump transferred to it. Soon
rwards a 10-in. pump was installed,
before the foundation level was
ched both pumps were running con-
ously at their full capacity. An
pump was kept in readiness as a
d-by. All the pumps were of the
rifugal type.

The bracing of the cofferdam was
posed of pitch-pine logs having
ntlings of from 11½ to 12½ in. square.
various tiers were connected to-
ter by 11 in. by 3 in. lacings, and the
ole suspended by loops of old wire-
e, passing round the walings of the
tier and over short pieces of iron rod
ssed through the holes in the pile-
ps used for attaching the driving-cap.
loops were tightened by twisters.
provide against the loosening effect
e to the intermittent load of the
ing and falling tide upon the piling and
acing of the river arm, the two sides
the piling were kept hard up on the
acing timbers by five 1-in. diameter
el tie-rods which extended the full
dth of the arm at high-water level,
ssing through the steel piling on each
le and through longitudinal walings on the outside. As a further provision against this intermittent load diagonal struts were fitted in the various tiers of bracing at the junction between the main trench and the river arm, while the outer end of the river-arm piling was strutted against the river staging.

As the transverse struts of the main trench were 45 ft. long, while the bays of timbering were about 8 ft. square, it was realised that considerable difficulty would be experienced in threading the struts of the lower tiers of bracing into position. A model of the complete bracing of the cofferdam was therefore made, and as the result of tests the transverse struts of the three uppermost tiers were made in one piece, while those of the two lower tiers were in two pieces, the joint having two half-timber fishplates with through-bolts. Before setting the timbers of the second tier of bracing the excavation was carried about 15 in. lower than was necessary for placing them. The walings and transverse struts of the third tier were then laid down, and on top of them those of the second tier were laid down and tightly wedged-up in place. As the excavation

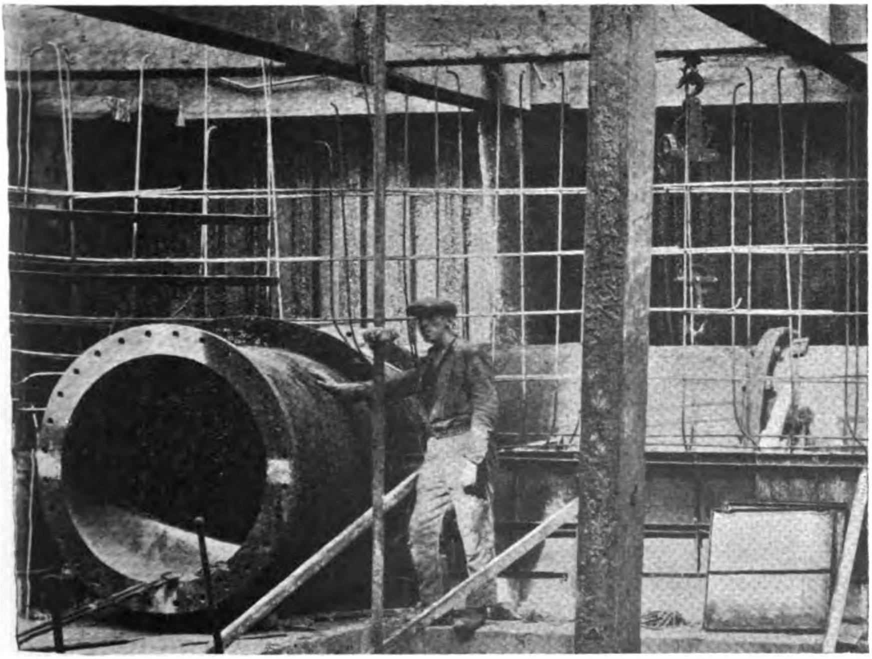


FIG. 8.—FORTY-TWO-INCH SUCTION PIPES READY FOR ENCASEMENT WITH CONCRETE.

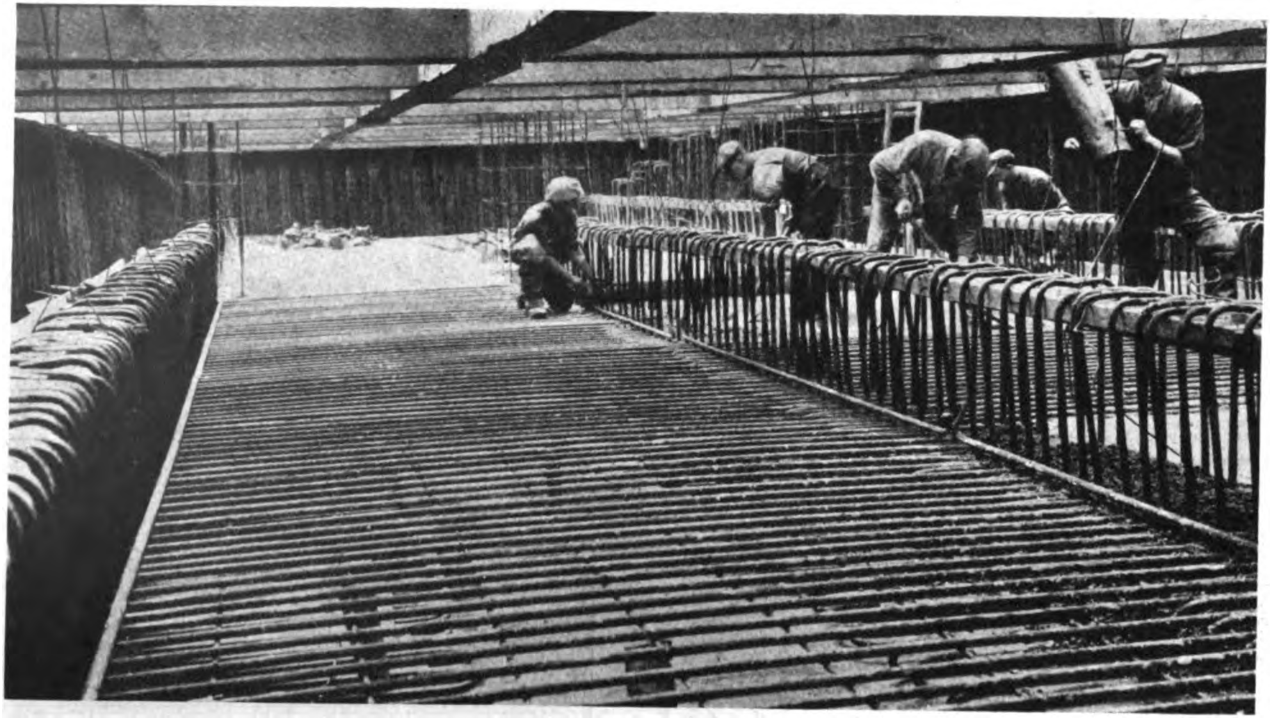


Fig. 9.—Concreting Portion of Floor of Inlet Culvert.
REINFORCED CONCRETE PUMPING-STATION AT ABERDEEN.

proceeded the timbers of the third tier were carried down till they reached their proper position, when they in turn were wedged up.

Before the reinforcement of the concrete floors of the pump and sump chambers and the inlet culvert was placed, the soft portions of the ground were removed and the springs led into a system of open-jointed pipe-drains. These were covered by a 12-in. layer of broken stone and gravel, which, in turn, was paved partly with a 3-in. layer of concrete *in situ* and partly with concrete slabs.

The presence of the bracing timbers, and the objection to replacing them by temporary struts bearing against thin concrete walls—especially while the walls were in a green condition—caused a very considerable amount of difficulty in the placing of some of the steel reinforcement, the arrangement of the concrete forms, and the placing of the concrete. The lowering and setting in place of the various lengths of the 42-in. diameter suction-pipes—especially the bends—was also somewhat troublesome. *Fig.* 8 shows some of those pipes ready for encasement with concrete. *Fig.* 10, taken after the laying of the concrete floor and during the removal of the lowest tier of bracing in the main trench, shows the small amount of headroom available when placing the steelwork and concrete of the floor, and also shows the temporary deformation of some of the wall splice-bars thereby necessitated. In the picture can be seen one of the transverse-strut timbers in process of removal by

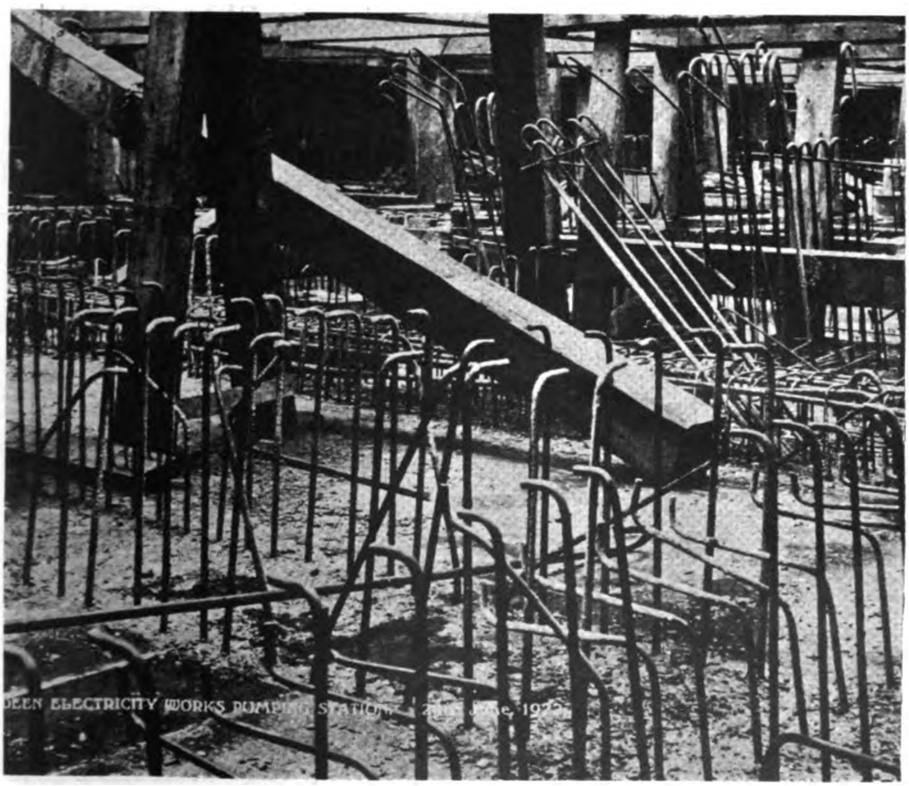


FIG. 10.—SHOWING TEMPORARY DEFORMATION OF WALL SPLICE-BARS NECESSITATED BY SMALL AMOUNT OF HEADROOM AVAILABLE.

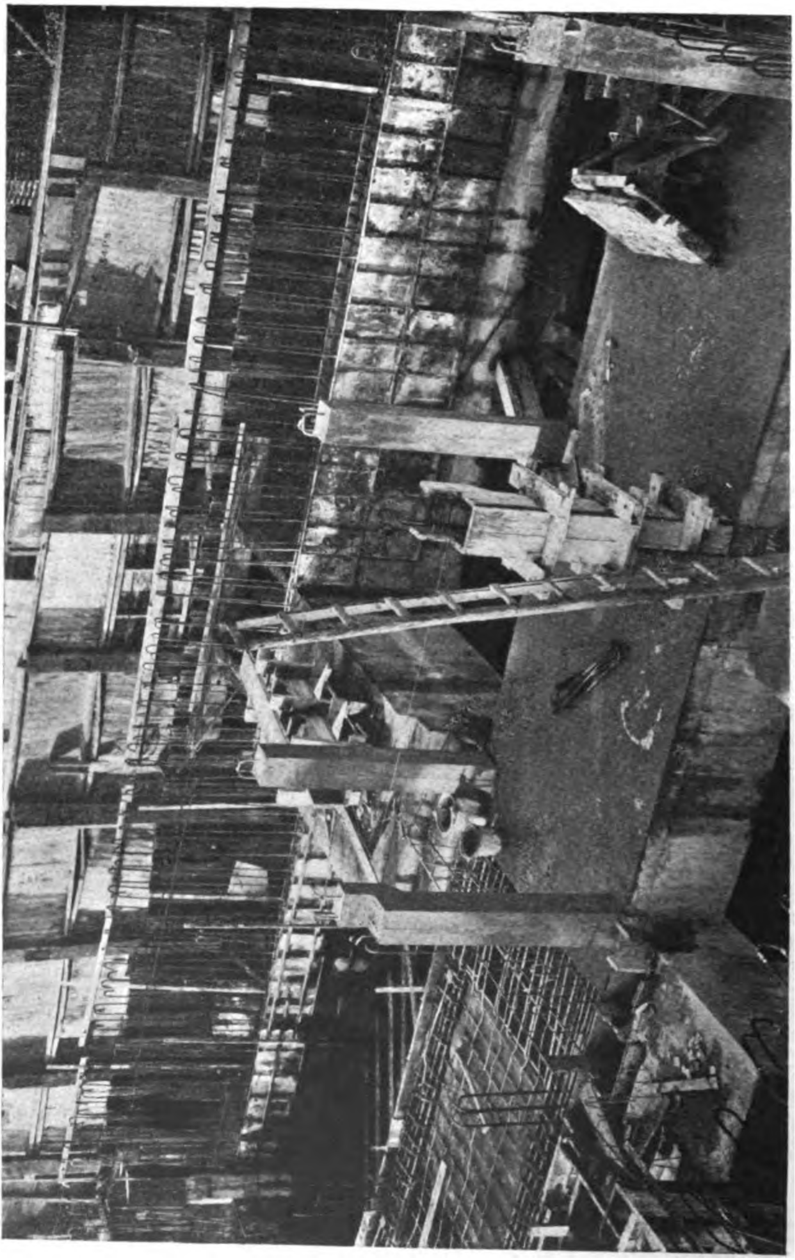


Fig. 11.—Partially Completed Portion of Gangway and Roof Columns of Suction-Chamber.
REINFORCED CONCRETE PUMPING-STATION AT ABERDEEN.

crane, the upper four tiers of bracing being meanwhile supported by temporary props as shown.

Before removing a tier of the bracing timbers the concreting of the walls was carried as high as possible, in lifts of 2 ft. at a time. The refilling round the outside of the buildings was also brought up as far as possible, its height being limited, however, by the fact that the upper tier of steel concrete-forms had to be left in place for the attachment of the following tier. Difficulty also arose with regard to the refilling of the sump, as it was important that the circular portion of the pump chamber, with its thin walls, should not be subjected to unbalanced external pressure. Timber casings, about 5 ft. by 5 ft. inside, were therefore built round the suction pipes of the pumps. A 2-ft. layer of gravel was laid to convey the drainage water from the river arm into the sump, and the portions of the sump outside the suction-pipe casings were also filled with gravel to the same level. Above this level ordinary filling was deposited, the timber casings being continued upwards. When the refilling had been completed up to 22 ft. below ground-level the three pumps were removed and the inside of the suction-pipe casings filled up. As much of the timber of the casings as could be easily taken out was removed, and the rest buried. After its removal from the main sump the 6-in. pump was again set to work in its original position at the outer end of the river arm.

Steel concrete-forms were used throughout, the proper thickness of the walls being determined by machine-made wire-ties keyed to the shuttering, and by temporary wooden struts which were knocked out when the concrete came up to their level. The ties were embedded in the concrete, their projecting ends being snipped off after the removal of the forms. The component parts of the forms were connected together by tapered keys, and milled clay was rubbed into any joints that were unduly open. Specially-curved stiffening angle-bars were required for the cylindrical portion of the pump chamber, but the standard 2-ft. by 2-ft. flat forms were used, the result being most satisfactory. Great care was taken to set the forms truly circular in this part of the work. To facilitate this a vertical iron-pipe was used as a centre round which a trammel could be rotated. The pipe extended the full height of the cylindrical portion of

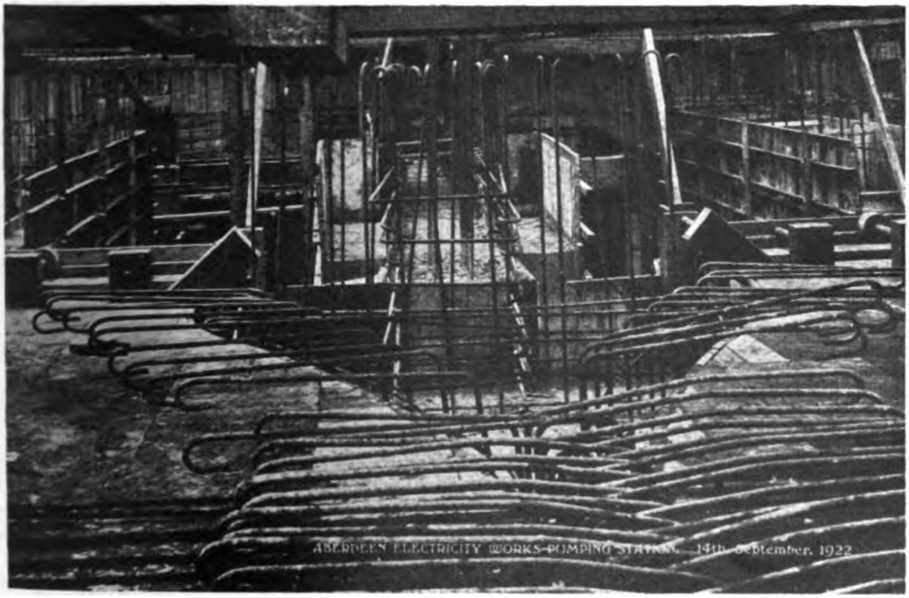

ABERDEEN ELECTRICITY WORKS—PUMPING STATION. 14th September, 1922.

FIG. 12.—PARTLY COMPLETED CENTRAL DIVISION WALL OF SUCTION-CHAMBER: SHOWING POINT WHERE TWO SECTIONS OF THE INLET CULVERT DIVERGE BEFORE ENTERING THE SUCTION-CHAMBER.

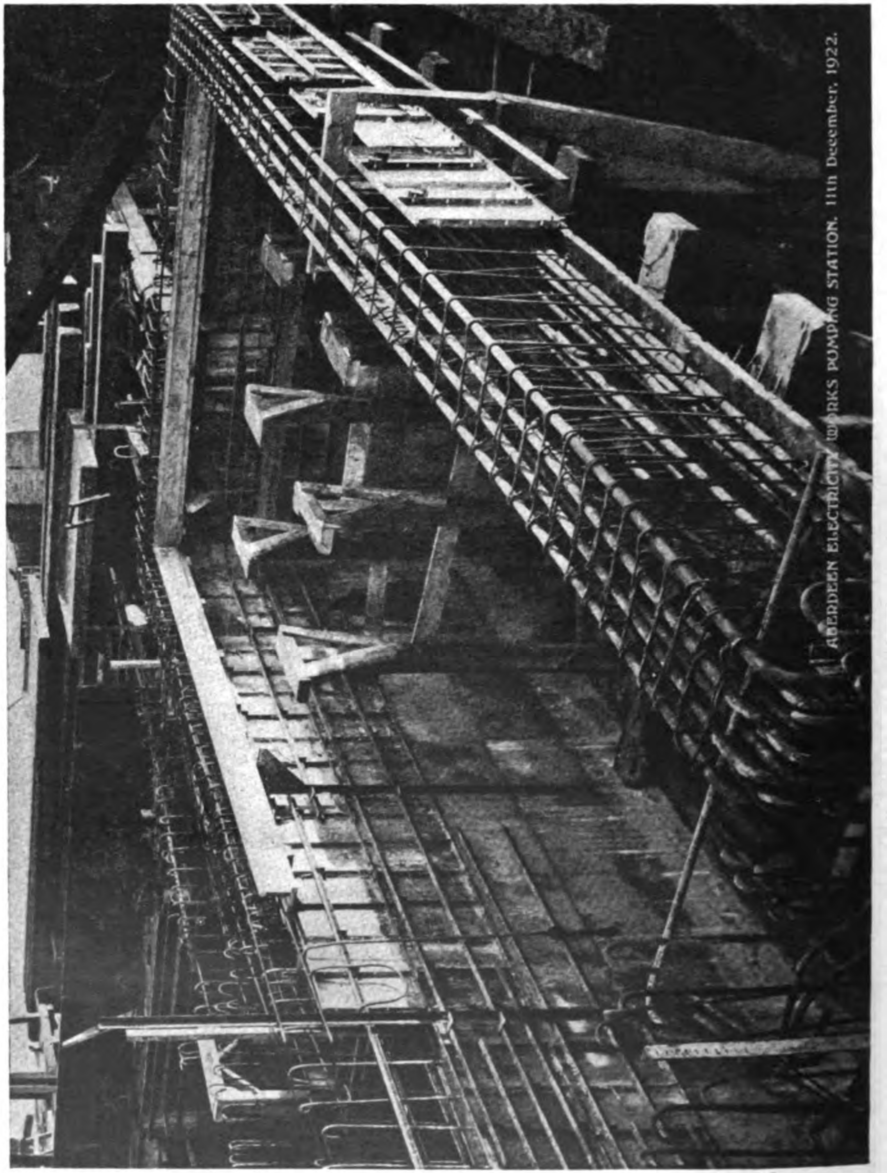

ABERDEEN ELECTRICITY WORKS POMPING STATION. 11th December, 1922.

Fig. 13.—Reinforcement of the Main Roof-Beam of Pump-Chamber.
REINFORCED CONCRETE PUMPING-STATION AT ABERDEEN.

the building. It rested in a footstep on the concrete floor, and at the top was secured in a bearing attached to the bracing timbers of the cofferdam. A clamp in two parts was so arranged that it could be fixed to the pipe so as to support the end of the trammel at any level. The trammel consisted of a tapered pole having at the thick end a deep slot, into which the vertical pipe fitted, while a cotter-pin kept the trammel firmly in position against the pipe. The smaller end of the trammel had a conical point, into which was driven a large nail with a hemispherical head.

The reinforcement of the concrete was throughout of plain round steel bars, to British Standard Specification No. 15. The bars varied from ⅜ in. to 1¼ in. in diameter, and were cold-bent on the site.

The reinforced concrete was made of cement sand and crushed granite, in the proportions 1 : 1½ : 3. The unreinforced concrete used for embedding the 42-in. diameter suction pipes, for the layer under the reinforced floors, the stairs, and a few minor portions of the buildings was made of cement, sand and gravel, in the proportions 1 : 2½ : 5. To this about 12 per cent. of displacers was added where possible. The granite was of such a size as to pass through a ¾-in. square mesh, while the gravel was screened by a 1½-in. mesh. All the gravel and sand used for concrete making were obtained from the excavations.

The sand and stone for concrete making were measured in ordinary ½-yd. iron

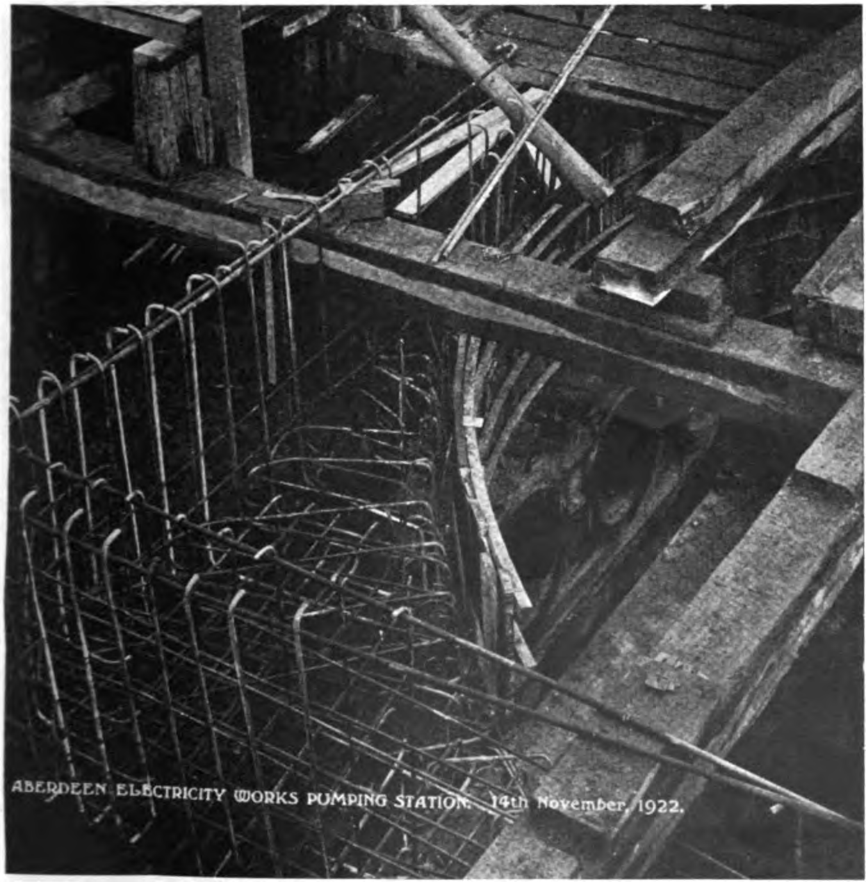

ABERDEEN ELECTRICITY WORKS PUMPING STATION. 14th November, 1922.

FIG. 14.—REINFORCEMENT AT CORNER OF PUMP-CHAMBER: JUNCTION OF CIRCULAR AND SQUARE SECTION OF THE BUILDING.

JAMES MITCHELL.

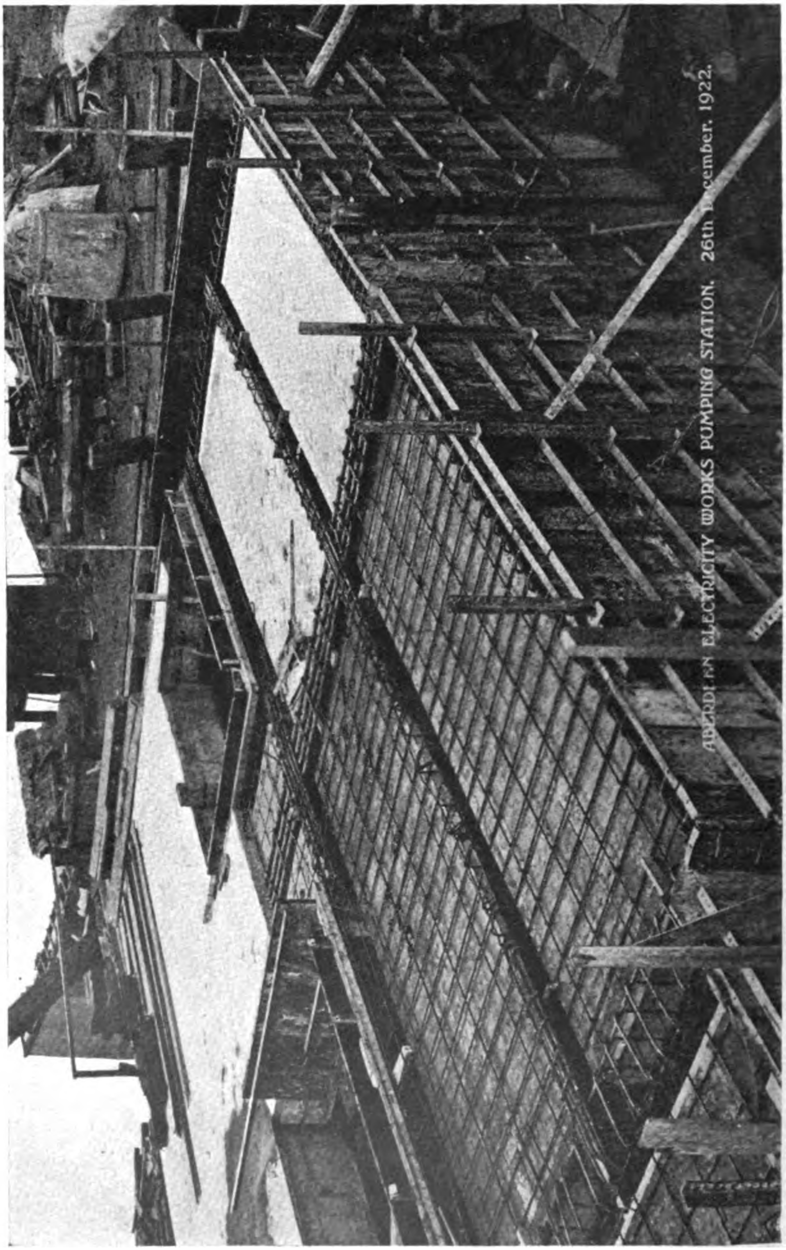

ABERDEEN ELECTRICITY WORKS PUMPING STATION. 26th December. 1922.

Fig. 15.—Partially Completed Roof, Showing Openings for Pavement Lights.
REINFORCED CONCRETE PUMPING-STATION AT ABERDEEN.

side-tipping wagons. The quantity of materials for one charge of the mixer was based on the volume of a bag-full of cement as received from the maker, so as to avoid the cost of emptying out and re-bagging the cement. A false wooden-bottom was put into the measuring wagon at such a height that the reduced volume, measured level with the top sides of the wagon, was equal to that of the stone, plus that of the sand. The height to which the stone had to be filled was indicated by the lower edges of bars riveted to the sides of the wagons. The remaining space was filled with sand, any excess being swept off by a screeding lath. The sand-pile was situated between the stone-pile and the mixer so as to minimise the travel of the measuring wagons. Two sets of measuring wagons were used, to suit the different proportions of the plain and the reinforced concrete.

The concrete was mixed by a ½-yd. Ransome mixer, and was conveyed to the different portions of the work by ½-yd. tipping-wagons running on a system of 2-ft.-gauge railways and turntables, laid on the top of the cofferdam bracing-timbers. From the wagons it was tipped into hoppers, attached to which were jointed tubes about 10 in. diameter. The hoppers and tubes were of thin galvanised-steel. They were easily handled by the 7-ton crane, and could be taken down and re-erected in a very short time. When the walls had reached such a height that depositing by this method was unsuitable the concrete was emptied from the mixer into iron boxes, which in turn were emptied on to timber platforms, or where the thickness of the walls permitted directly into the forms. From the platforms it was conveyed in buckets, or shovelled direct into the forms. Electricity was used for the driving of the concrete mixer and the drainage pumps, and for the lighting of the works.

In order to prevent the adhesion of the concrete from interfering with the withdrawal of the steel piling, rough boarding ⅜-in. thick was interposed between the two at the edges of the floors—see *Fig.* 7—and alongside the concrete surrounding

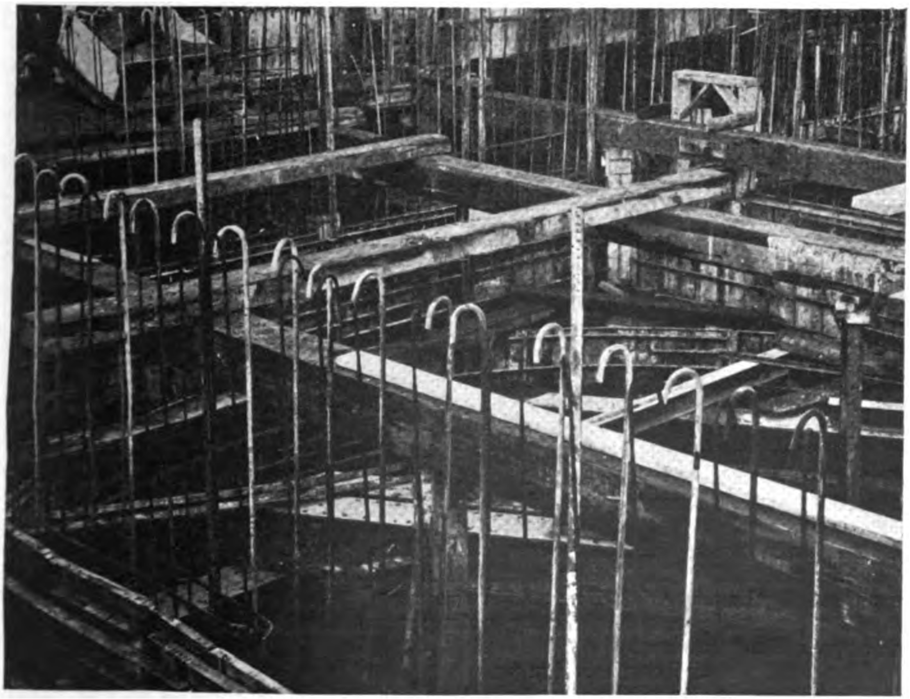


Fig. 16.—Showing Steel Beams for the Support of the Pump Motors.

the 42-in. diameter suction pipes. As a further precaution, the interlocking edges of the piles were coated before driving with a mixture of graphite and grease.

The roofs of the pump and sump chambers have large openings for the removal of the machinery, and these openings are fitted with pavement lights. A gangway above high-water-level extends the whole length of the sump chamber. It was necessary, however, to make independent entrances to the pump chamber and the sump chamber, through hatchways in the roof of the passageway between the two, as the necessary headroom under the roof would have brought the sill of a door communicating between the two chambers below the level of high tides, and thus exposed the pump chamber to flooding. As the roofs form part of a public footpath the concrete portion of them was finished with granolithic 1-in. thick. The whole of the interior surfaces of the walls were rubbed down with carborundum blocks, and then painted with two coats of cement grout. As a precaution against the percolation of water through the walls of the pump chamber they were coated on the outside with thick coal-tar applied hot.

Fig. 9 shows the concreting of a portion of the floor of the inlet culvert in progress, the splice bars of the centre and wing walls being shown in place. At the right hand a man is to be seen holding the lower end of the chute for depositing the concreting. *Fig.* 12 shows the partly-completed central division-wall of the sump chamber, and the point where the two sections of the inlet culvert diverge just before entering the sump chamber. A few of the cores for the bolt-holes of the cast-iron sluice-frames can be seen, and also a portion of the steel concrete-forms set in place for the culvert roof. *Fig.* 11 shows a partially-completed portion of the gangway and roof columns of the sump chamber. *Fig.* 14 shows the steel reinforcement of a corner of the pump chamber at the junction of the circular and the square sections of the building. The three 30-in. diameter delivery-pipes of the pumps can be seen. *Fig.* 13 shows the reinforcement of the main roof-beam of the pump chamber and the props set in place for the support of that of two of the subsidiary beams. In *Fig.* 16 can be seen the steel beams for the support of the pump motors. All the bracing timbers have been removed except the three shown, which were put in at the level of the top of the steel piling before removing the top tier so as to prevent movement of the foundation of the 7-ton crane. In *Fig.* 15 is shown the partly-completed roof, with the openings for two pavement-lights.

Mr. J. Alex. Bell, M.Inst.E.E., Electrical Engineer to the Corporation of Aberdeen, was responsible for the design of the works, and they were carried out under his supervision, in collaboration with Mr. Hugh R. Barr, Assoc.M.Inst.C.E. They were executed by Mr. William Tawse, public works contractor, Aberdeen, for whom the writer acted as agent and engineer.

Silos for Cement.

THE two wooden silos used for storing cement at the Klagstorps Kalkbrotts A.-G. in Sweden, having been destroyed by fire, have now been replaced by new ones built of concrete.

The silos are cylindrical; each is 53 ft. in diameter and 66 ft. high, the capacity being about 10,000 tons of cement. The conical portion must be particularly well built, and for this reason a strong ring is placed at the junction of the cylindrical and conical portions.

The walls are 4½ in. thick above and 18 in. below.

Rails are laid on both sides of the silos, so as to give direct access for loading purposes. Sacks are filled direct from the silos by means of "Excilors," so that instead of an hourly output of ninety sacks per pair of men, one man can now easily pack 150 sacks per hour.

The whole of the construction was completed within six months.

The sides were designed and patented by Messrs. F. L. Schmidt & Co., of Copenhagen and London.—*Beton u. Eisen.*

COMBINATION PRE-CAST AND *IN SITU* REINFORCED CONCRETE BRIDGE.

CONSIDERABLE attention has been called to the Wellesley Road Bridge over the L. & S.W. Railway, Gunnersbury, W.4, during and since construction, and when tests were made remarkable results were obtained. A short history and description of the reconstruction will therefore probably be of interest to our readers.

The original bridge was constructed 30 ft. in width between the parapets when the railway was built, with cast-iron girders and columns, the former filled in with jack-arches, and ordinary road construction carried out above. Apparently when the Richmond extension was carried out the bridge was widened with wrought-steel girders, in all cases the abutments adjoining the earth being of brick. There was insufficient cover for large gas or water mains, so that the 24-in. main of the Metropolitan Water Board was necessarily carried on girders outside the bridge on the northern side, and, therefore, any scheme for reconstruction of the bridge had to include provision for dealing with this main.

It will be seen from *Fig.* 1 that any widening of the bridge to give good alignment would be necessarily of variable widths on both sides, and in each case no uniformity as to width could be adopted. Furthermore, the Company would only recognise 5-ton loads over the old structure, and, in view of the heavy modern motor traffic, the Engineer to the Chiswick Urban District Council, Mr. Edward Willis, M.Inst.C.E., M.I. Mech.E., F.S.I., M.Inst.T., strongly recommended the Council to rebuild the bridge of at least 50-ft. width, subject to a proportionate payment being obtained from all persons or authorities affected thereby. He was therefore instructed by the Council to prepare designs and specifications and negotiate for such reconstruction, with the result that after some months this was agreed upon, and the Ministry of Transport, Middlesex County Council, London & South-Western Railway Company, and the Chiswick Council each agreed to pay 25 per cent. of the cost of the bridge, the contribution not to exceed £3,000 by any one of the contributing parties.

The London & South-Western Railway Company, as one of the contributors, requested the Engineer of the Council to act in association with Messrs. L. G. Mouchel & Partners so far as the ferro-concrete design was concerned, and in accordance with this arrangement Messrs. Mouchel prepared the ferro-concrete design, although certain modifications were adopted at the Engineer's suggestion.

The design for the bridge included partly pre-cast ferro-concrete beams, partly *in situ* ferro-concrete beams, brickwork wing walls, and extension of main abutments, with ferro-concrete columns as a central support to the main span, this being the first bridge of its type in the neighbourhood with such a combination.

Tenders were received from fifteen contractors, and the lowest, that of Messrs. Davidson & Sykes, Ltd., of 32, Victoria Street, Westminster, S.W.1, was accepted. This firm has now completed the work and it is anticipated the total cost will be below the original estimate. The actual cost of the Metro-

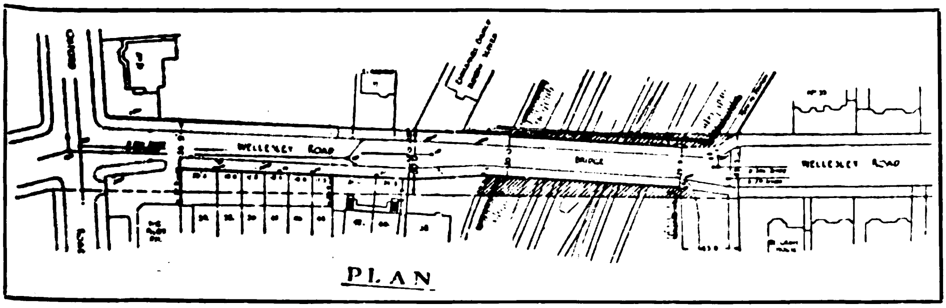
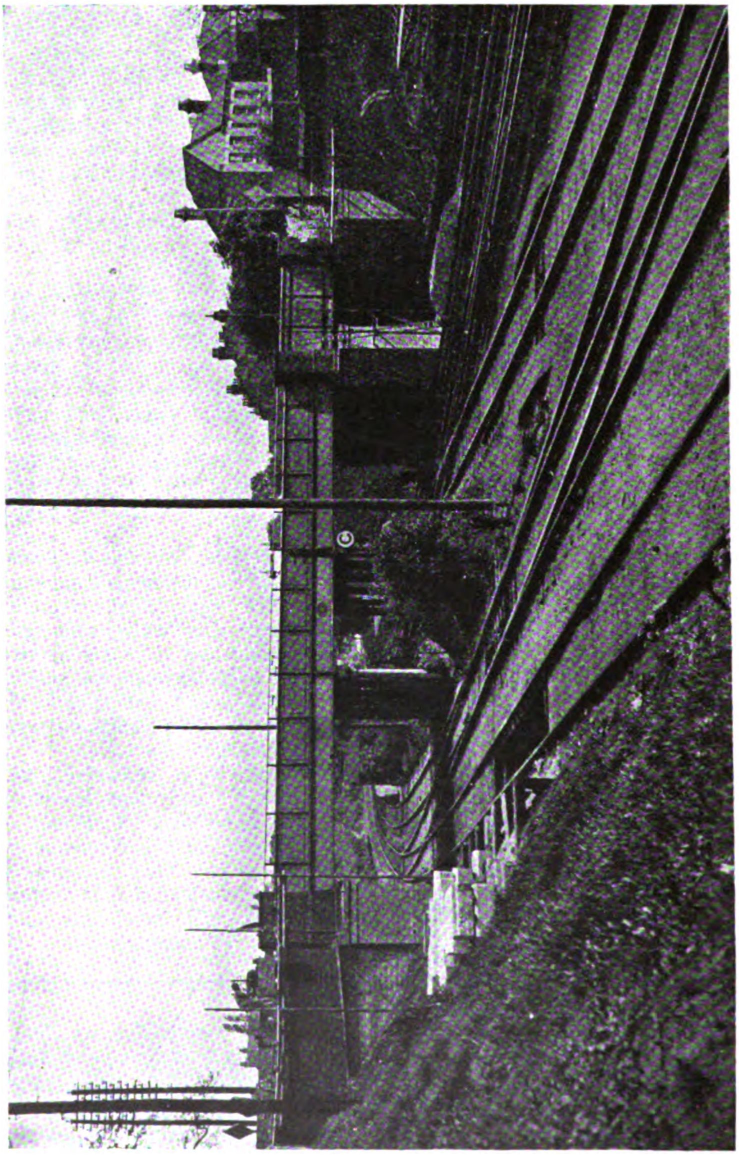
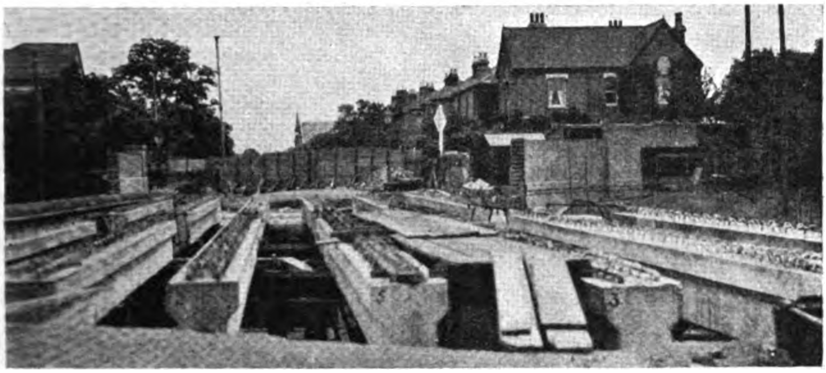


FIG. 1.—WELLESLEY ROAD BRIDGE, GUNNERSBURY.



Wellesley Road Bridge, Gunnersbury.



Wellesley Road Bridge, Gunnersbury.

politan Water Board's alterations, and the relaying of the water main in a special channel under the foot-path, is expected to be over £800.

The new bridge measures 97 ft. in length between the faces of the abutments and 50 ft. between the parapets, this giving an increased width of nearly 20 ft., which has enabled the Engineer to provide a roadway 32 ft. wide and two footpaths 9 ft. wide. The minimum height of 14 ft. from rail level to the underside of the bridge was necessary to meet the requirements of the London & South-Western Railway Company over whose lines it passes, a condition which was fulfilled without difficulty by the ferro-concrete construction.

The existing abutment walls have been left in place and extended to conform with the greater width of the new bridge. The two intermediate lines of cast-iron columns have been entirely removed and replaced by a single line of ferro-concrete columns, moulded *in situ* and rising from extended bases, 2 ft. 3 in. below rail level.

The columns are 18 in. sq. in cross section, and the bases are 6 ft. 9 in. sq. for the three inner columns and 4 ft. sq. for the two outer columns. The latter members were prepared for monolithic connection with a line of main beams extending from side to side of the bridge, and moulded *in situ*. The two inner beams measure 16 in. wide by 2 ft. 9 in. deep, each of the outer beams being of the same width by 3 ft. deep below ducts. The beams were constructed with recesses for the reception and support of pre-cast longitudinal beams, extending between the abutment walls at each end and the line of columns in the centre of the bridge. In order to stiffen the construction, diagonal bracings have been provided between the two outer columns and the columns next to them.

Along each side of the bridge extends a continuous ferro-concrete beam, 14 in. wide by 4 ft. deep, supported by the abutment walls and the outer columns and in monolithic connection with the latter. These beams are continued upwards in the form of panelled parapet walls, with a maximum thickness of 10 in. and a height of 4 ft. 10 in., the parapets acting with the beams beneath them and contributing materially to the strength and rigidity of the construction.

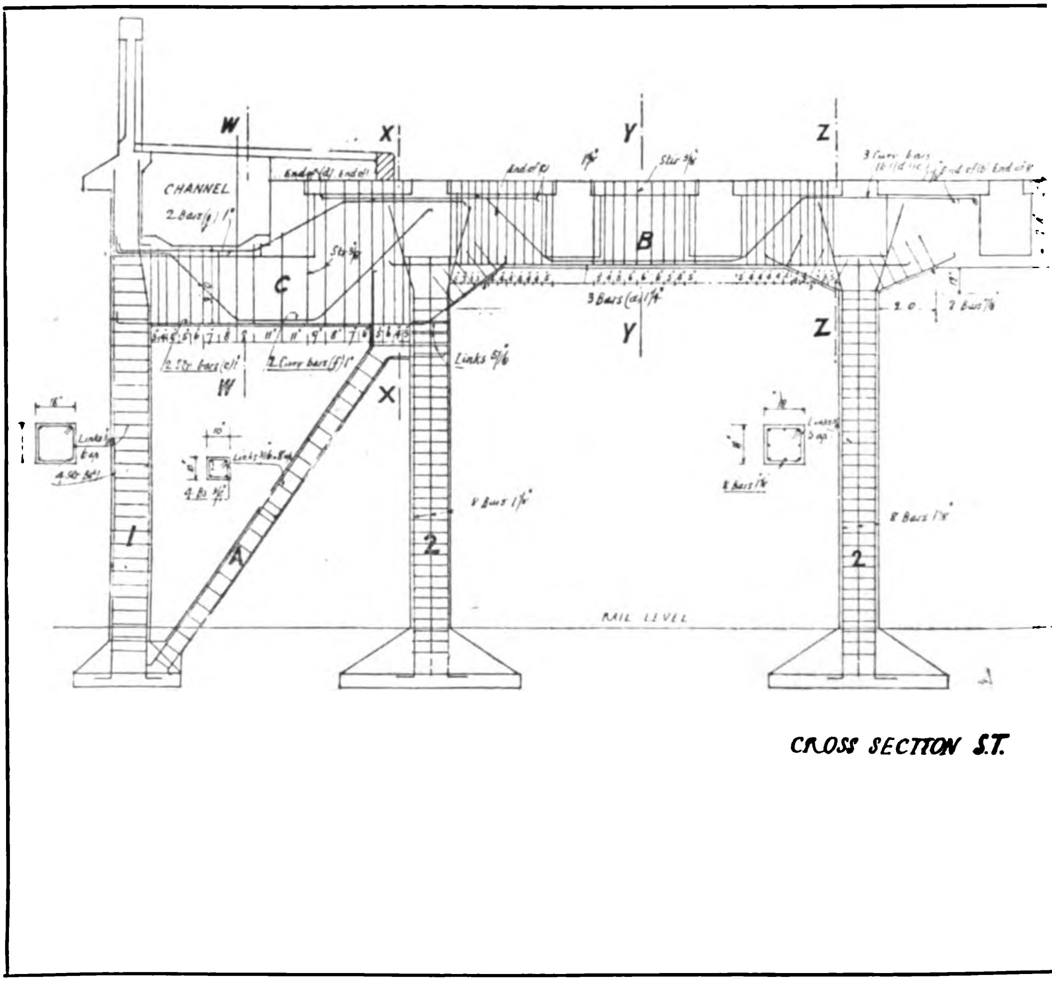
The pre-cast longitudinal beams previously mentioned were moulded in a yard established and equipped by the contractors close to the site of the work, and after having sufficiently seasoned, were removed and placed in position by a powerful travelling crane hired from the London & South-Western Railway Company. The pre-cast beams are 28 in. deep by 14 in. wide at the bottom, the top being widened to 22 in. so as to afford ample support for the pre-cast slabs, 6½ in. thick, which constitute the decking of the bridge. The beams were reinforced in accordance with standard Mouchel-Hennibique practice, specially arranged reinforcing bars projecting from the top and ends as required for insertion in the concrete of the main transverse beams, and for incorporation in the concrete placed to form connections between the beams and decking slabs. The beams are spaced 5 ft. apart and the slabs are 3 ft. 10 in. wide, thus allowing a space of 14 in. for the *in situ* connections.

York stone bedplates were placed and fixed on top of the extended abutment walls, and to them the pre-cast beams were securely connected. Channels have been formed beneath the footpaths on each side of the bridge, the bottom 4 in. thick and the top being formed by the paving slab of the footpaths, these also being 4 in. thick.

The combination of pre-cast and *in situ* work embodied in this bridge was introduced some years ago by Messrs. L. G. Mouchel & Partners, with the object of reducing the time necessary for the construction of ferro-concrete bridges, a noteworthy example being the reconstruction of St. John's Hill Bridge, over the London & South-Western Railway, for the London County Council.

LOAD TESTS.

The bridge was designed for a rolling load on the carriageway of 50 tons on



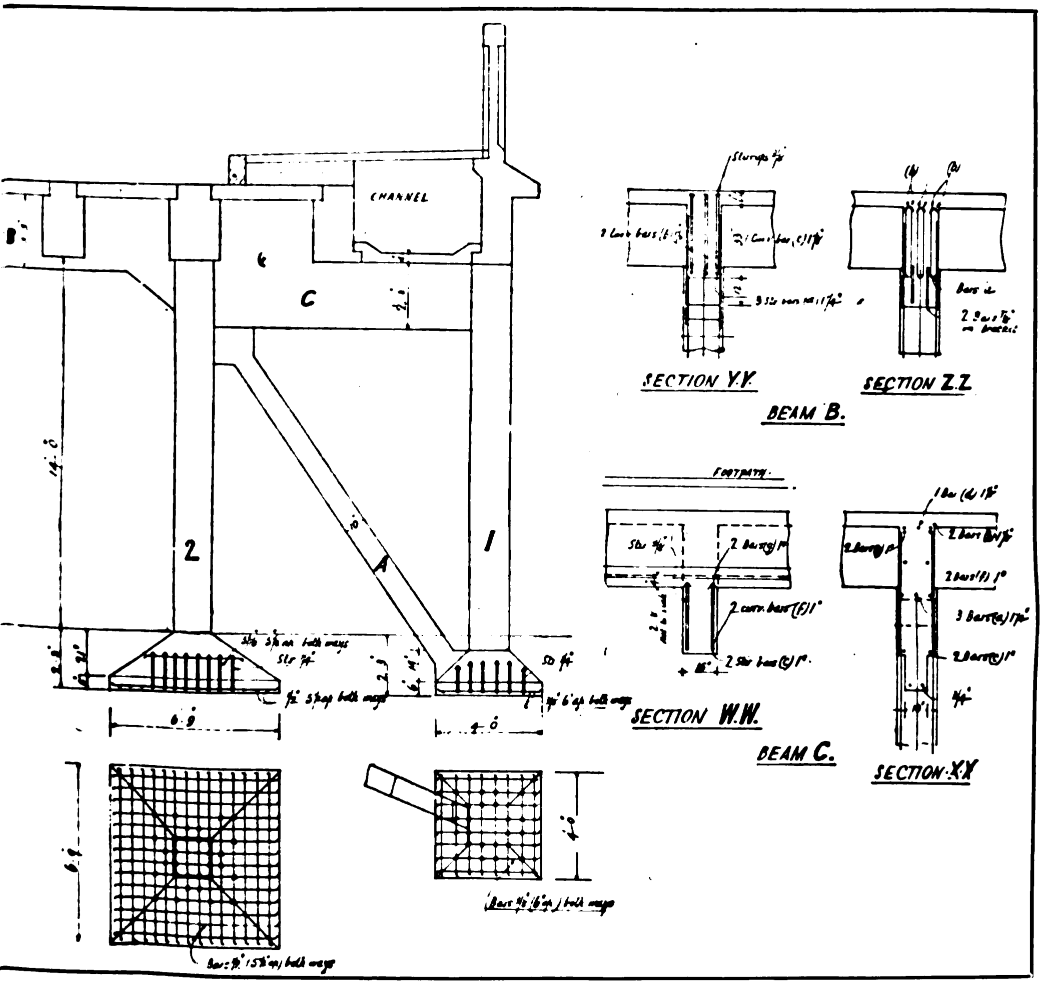
COMBINATION PRE-CAST AND "IN SITU" BRIDGE, GUN
(A Drawing showing the Second

four wheels and a superload of 1 cwt. per sq. ft. on the footpaths. On November 21, 1922, official tests were conducted in the presence of the Engineer (Mr. Edward Willis), Messrs. Bond & Pimley (of the London & South-Western Railway), Mr. Bass (representing Mr. A. Dryland, the Middlesex County Surveyor), Mr. C. R. J. Wood, A.M.Inst.C.E. (Director of Messrs. L. G. Mouchel & Partners), Major Davidson (representing the contractors), and Mr. Williams (clerk of works).

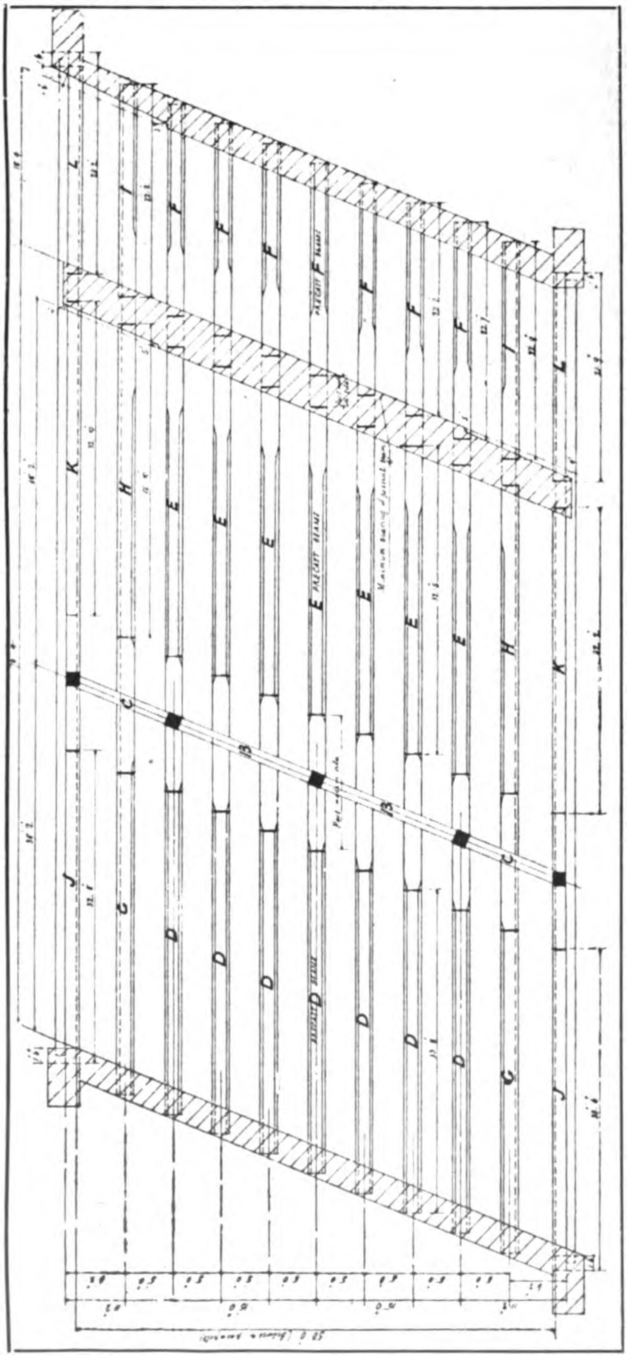
The following rolling loads were used :—

1	steam roller	weight	11	tons	15	cwt.
1	,, ,,	,,	13	,,	15	,,
1	petrol roller	,,	8	,,	17	,,
1	loaded wagon	,,	12	,,	0	,,
1	,, ,,	,,	12	,,	0	,,
1	,, ,,	,,	14	,,	10	,,
1	,, ,,	,,	11	,,	1	,,
	Total .	.	83	tons	18	cwt.

Measurements of deflection were taken by means of deflectometers, one placed under the centre of the main beam carried by the line of ferro-concrete columns, and three others placed under the end span

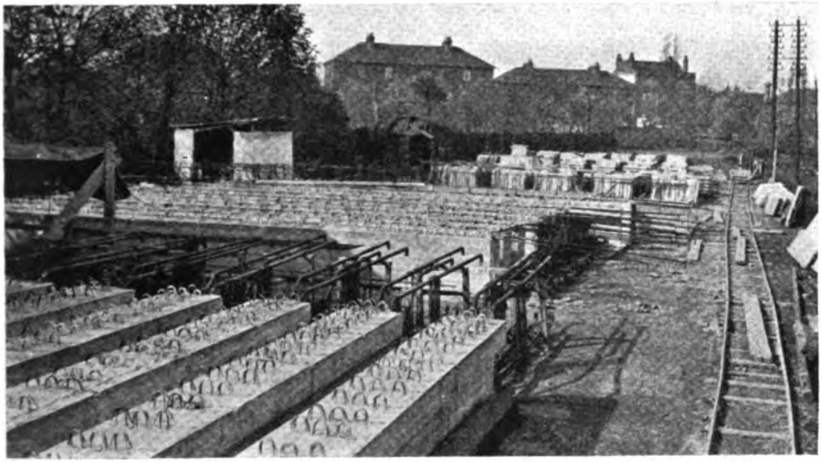


SBURY : Slabs, Columns, Struts and Main Beams.
y Beams is given on pp. 188–9.)



General Plan showing Pre-Cast Beams.

COMBINATION PRE-CAST AND "IN SITU" REINFORCED CONCRETE BRIDGE AT GUNNERSBURY.



Wellesley Road Bridge, Gunnersbury.

of 38 ft. 2 in. between the line of columns and the adjacent abutment. Of these instruments, one was placed under the centre pre-cast beam and the others under two beams next but one to the centre beam.

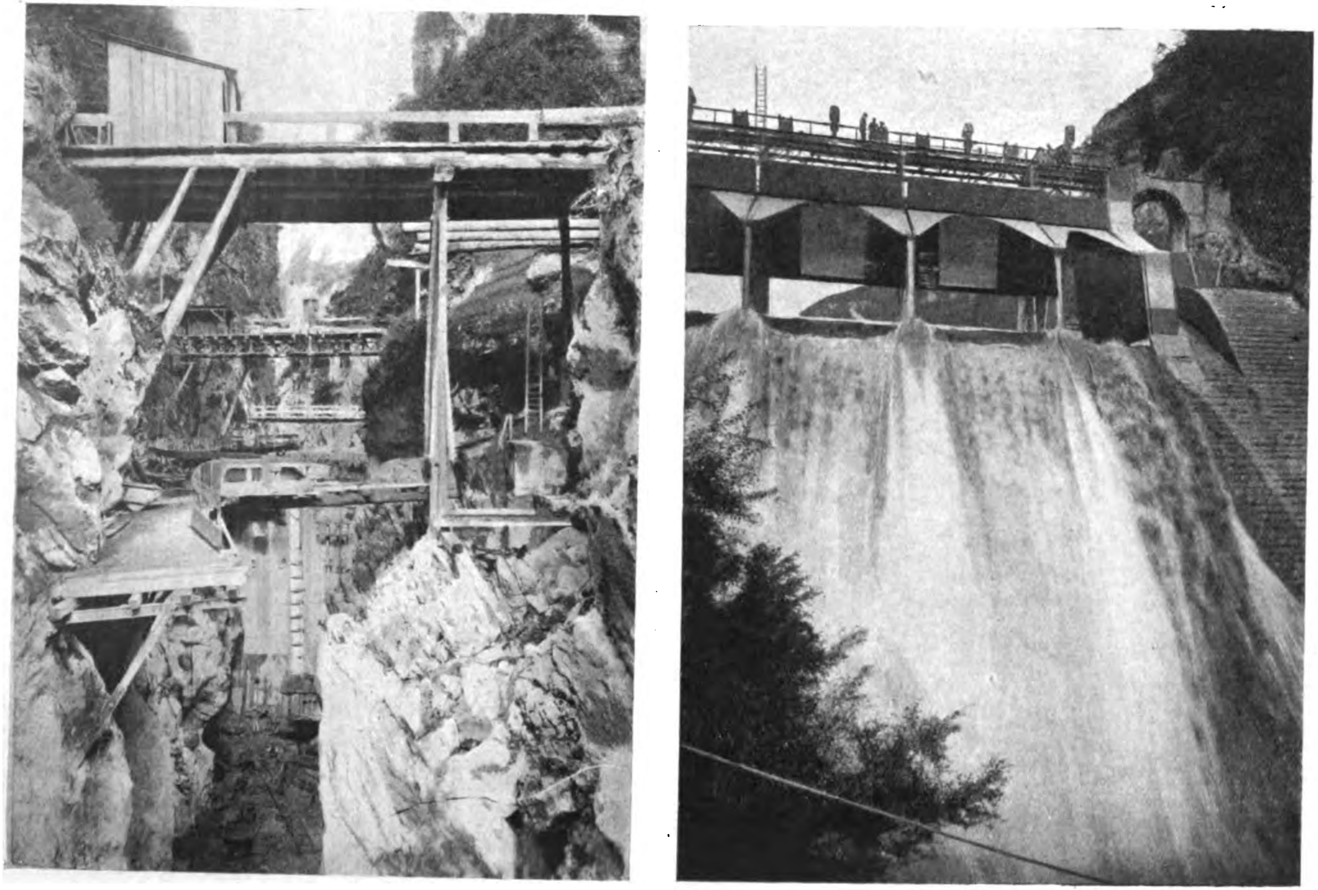
Test No. 1.—In this test, the three rollers were run in single file along the centre of the span and brought to a stand as close together as possible, and two loaded wagons were then brought up on each side of the rollers, the total load on the span then being 83 tons 18 cwt. The maximum deflection, in the centre beam, amounted to 3½-64ths of an inch, and after the loads had been run off, a permanent set of ¾-64th of an inch was registered.

Test No. 2.—In this test the rollers were run over the bridge in single file along the centre line, and the loaded wagons were run across in the opposite direction, two on each side of the line of rollers. All the vehicles were run on and off the bridge without stopping, with the object of producing as nearly as possible the conditions of maximum loading obtaining in the conduct of ordinary traffic. The maximum deflection recorded was 1½-64ths of an inch. The maximum deflection attained, viz., 3½-64ths of an inch, represents 1-8375 of the span, as compared with 48-64ths of an inch, or 1-600 of the span, permissible under the specification.

The large margin of rigidity over that provided for in the specification points to the satisfactory character of the work, the strength of which will continue to increase from year to year with the progressive hardening of the concrete.

The concrete slabs were covered with *in situ* concrete to a camber of approximately 1 in 60, and this was again covered with 2 in. of asphalte incorporated with 25 per cent. of fine granite, laid by the Whitehall Asphalte Company, the remainder of this road having been similarly coated by this Company above the existing macadam road surface.

The appearance of the bridge is enhanced by the design of the moulded coping and pannelling, the surface of the granite concrete being roughened, and the Leicestershire facings to red brick piers and carved wing-walls with pressed blue brick plinths, and Bramley Fall stone caps and bull-nosed blue brick copings. The approach gradients are practically eliminated.



DURING CONSTRUCTION.

THE COMPLETED DAM.

THE DAM AT THE PORTES DU FIER. (*See p.* 177).

THE DAM AT THE PORTES DU FIER.

A DAM had to be built by the Lyons' Hydraulic Co. (Société Hydro-Électrique de Lyon) across the river Le Fier, which is in the nature of a torrent and has a delivery varying from about 10 cubic metres (13 cubic yards) at the lowest water-level up to 1,000 and 1,200 cubic metres (1,308 and 1,569 cubic yards) in flood tides. The retaining altitude, viz., the banked-up water-level, had to be about 36 metres (118 ft.).

The following solution of the problem was adopted :—

(1) Tunnels were driven in the steep banks (Portland limestone) on each side of the river.

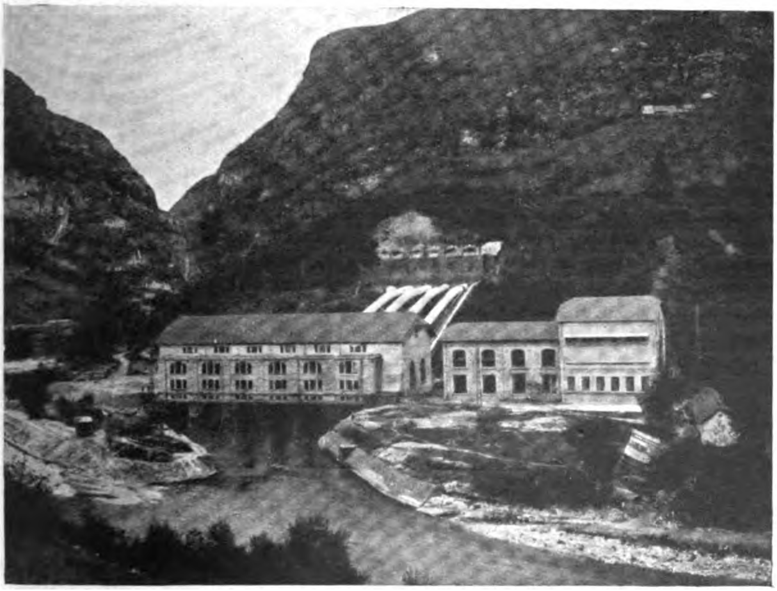
(2) The site of the dam was isolated by two cofferdam caissons—one on the up-stream side and the other on the down-stream side of the dam to be built.

(3) The water was drained from the enclosure thus formed, and the dam built with its foundation on the natural rock of the river bed.

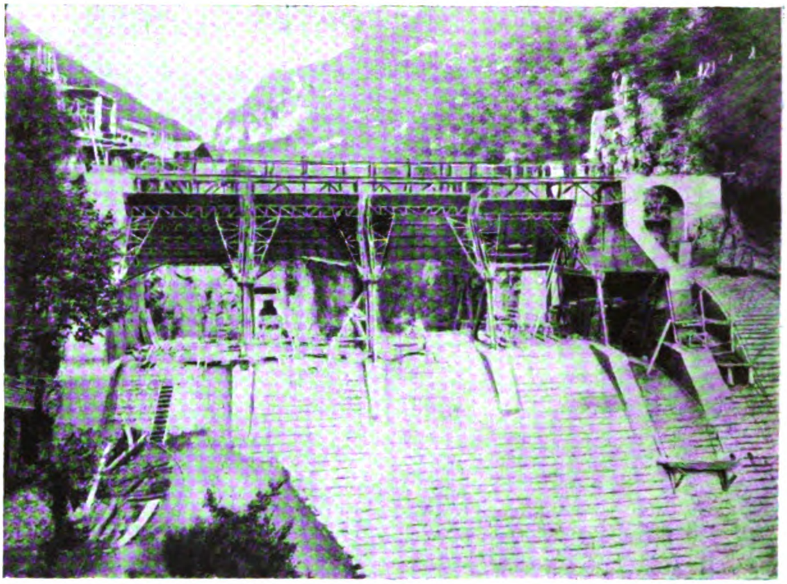
The preliminary work of boring to the right of the site of the caissons and the dam revealed a sharp incline rising from the up-stream to the down-stream side. Rock was struck on the up-stream side at an altitude of about 229 metres, whereas the average altitude was about 235 metres at the middle of the work, and 241 metres on the right of the site of the down-stream caisson.

The branch tunnels, each with a section of 30 sq. metres (322·9 sq. ft.) having been driven both on the right and the left bank, the work of lowering the caissons was commenced. The caisson for the down-stream side was about 18 metres (59 ft.) long over the width of the river, and 5·50 metres (18 ft.) deep. Sinking was continued from the end of 1912 until September, 1913, and the excavations made by blasting with dynamite were stopped at an altitude of 241 (16 metres) below low-water level, whereas the rocky bed had been struck from one bank to the other at about the altitude 242.

The work of lowering the caisson on the up-stream side was continued from altitude 258 metres to about 238. At this depth the caisson became wedged and could not be lowered further. As the rocky bed had not been on all the length of the working place, it was



THE DAM AT THE PORTES DU FIER: GENERAL VIEW.



THE DAM AT THE PORTES DU FIER: THE SLUICES.

decided gradually to continue the sinking while injecting cement into the gravel. The deepening of the working chamber was thus continued in cement concrete whilst the supporting ridge was gradually lengthened with iron and cement walls, which thus formed the sides of the working chamber.

Thus by successive injections underneath these walls it was possible to descend from the altitude of 238 metres to 229·80, the low-water level on the upstream side being then at an altitude of about 360 metres.

After taking every precaution to assure, as far as possible, the impermeability of the caissons, the work of draining off the water and the removal of the gravel was commenced.

The war put an end to work for nearly four years, and the great floods during this period again inundated the enclosure made by the caissons, so that drainage operations and removal of the gravel had to be done over again. Finally, in December, 1917, the building of the dam was commenced, and this was completed at the beginning of 1920.

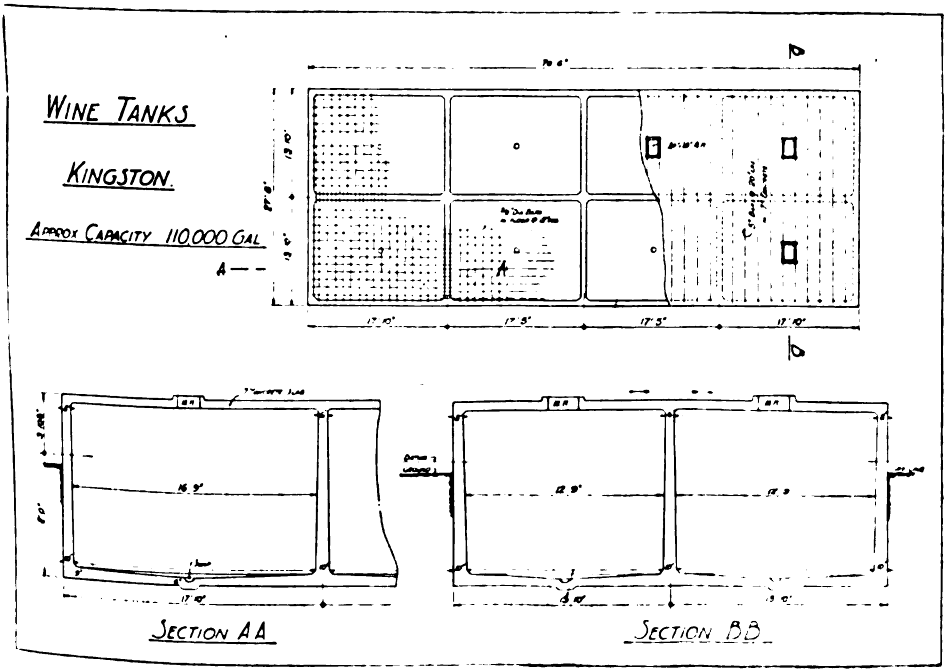
The crest of the dam is surmounted by five large segment-shaped sluice gates of the "Bosshard" type, and this system for discharge of flood water is completed by a lateral outflow tunnel regulated at the head by a Stoney gate (8 by 6 metres). The water thus raised is conveyed through a tunnel with a useful section of 32 sq. metres, driven through the mountain and lined along the full length of 1,400 metres. At the end of the tunnel there is the water chamber, from which run four mains, about 100 metres long and 2·60 metres in diameter. A fifth main is to be laid for a future enlargement of the works.

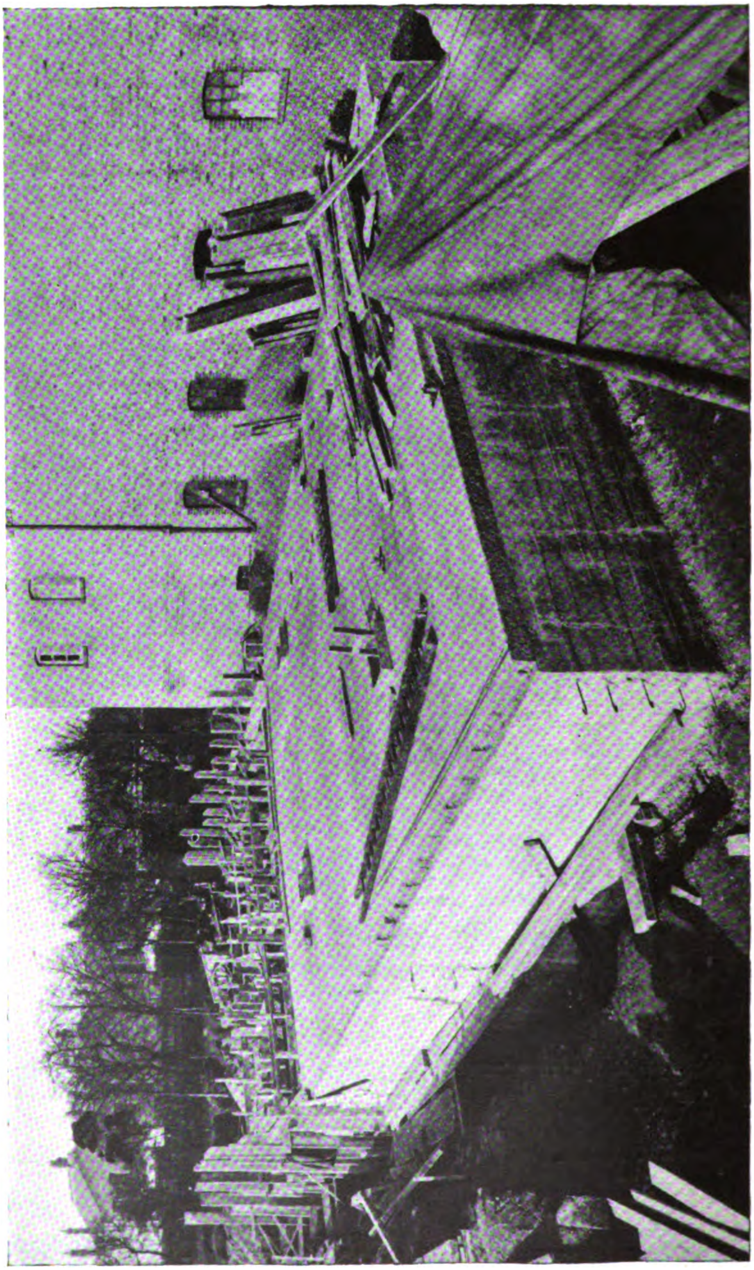
REINFORCED CONCRETE WINE TANKS.

THE tanks illustrated herewith are now nearing completion at Kingston-on-Thames for a firm which has been engaged in the production of medicated and other wines for a number of years. Some twelve years ago a series of reinforced concrete tanks, each of about 14,000 gallons capacity, was put down in which to store and ferment wine, and the whole of these tanks, with the exception of one, are giving complete satisfaction to-day and have given no trouble whatever since they were first put into use. The exception is a tank built separately from the others by a different firm of contractors, and we are informed by the proprietors of the works that this tank did not receive the same careful attention during construction as the others. It is, in fact, the exception which proves the rule that with proper workmanship to ensure a dense and sound structure concrete is unaffected by the action of wine during fermentation. It is a further testimony to the suitability of the material that the owners, after twelve years' experience with reinforced concrete tanks, are now having a further series of eight tanks built and propose shortly to proceed with another eight.

The new structure is divided into eight separate tanks divided from one another by partition walls, each tank measuring 16 ft. 9 in. by 12 ft. 9 in., by 11 ft. deep. The floors are 6 in. thick in the centre, tapering to 9 in. at the sides, and are reinforced with ⅜-in. diameter bars spaced at 12-in. centres. Sump-holes, 8 in. in diameter by 3 in. deep, are provided in the centre of each tank. The walls (both outside walls and partition walls) are tapered from a thickness of 10 in. at the bottom to 6 in. at the top, and are reinforced horizontally with ⅜-in. diameter bars under each face leaving a cover of ⅞ in. of concrete on both sides; these bars are at 3-in. centres at the bottom of the wall, and the spacing increases to 7-in. centres at the top. The vertical reinforcement consists of ⅜-in. diameter bars at 12-in. centres.

In view of the fact that heavy vats are to be carried on the top of the tanks, the roof slab was designed to take a load of 5 cwt. per sq. ft. It is 7 in. thick, and is reinforced with 5-in.





Reinforced Concrete Wine Tanks at Kingston.

" Feresco " bars at 18-in. centres. Manholes with iron covers are provided in the centre of the top of each tank to give access for cleaning purposes. All the corners are provided with extra heavy reinforcement, of ⅝-in. and ⅜-in. bars, in order to obtain an absolutely rigid connection, and negative bending moments at fixed ends have been provided for.

Each tank has a capacity of 14,240 gallons, and it is the practice to keep them full, the wine sometimes extending half-way up the manhole. They are sunk to a depth of 8 ft. below ground, and 3 ft. shows above ground.

In the construction of these tanks the contractors have taken special care to obtain impermeability to prevent losses of wine and to get a smooth, hard surface which will not injure the wine by crumbling or dusting. The footings of all the walls are splayed and all corners are rounded off. The concrete consists of 1 part cement (" Ferrocrete "), 2 parts sand, and 4 parts ballast (supplied by the Ham River Grit Company). The inside of the tanks has been rendered with ¾ in. of sand and cement, applied in two thicknesses of ⅜ in., trowelled to a smooth surface, and in placing the concrete between the forms care is taken that it is well stamped down in order to make it as dense as possible.

The design and construction of the tanks is being carried out by the London Structural Company, of Stuart House, Tudor Street, E.C.

[In our issue for March, 1922, we illustrated and described some reinforced concrete tanks for the fermentation of beer.]

BOOK REVIEW.

Mechanical Testing. Volume II. Testing of Prime Movers, Machines, Structures and Engineering Apparatus. By R. G. Batson, M.Inst.C.E., and J. H. Hyde, M.Inst.C.E. (both of the National Physical Laboratory).

(London : Chapman and Hall. Price 25*s*. net.)

It is a matter for regret that structures are not tested more frequently. The mechanical engineer obtains important data from the careful study of results of periodic tests of engines or machines, and in the same way the structural engineer could obtain extremely important information about structures working under service conditions. Designers may or may not appreciate certain aspects in the problems before them, but if definite data, obtained by measurements taken under working conditions, were available then the resulting structures should be the more economical and efficient.

On this account the second volume of *Mechanical Testing* is very welcome. The book is mostly devoted to measurements belonging to mechanical engineering, dynamometry, gear testing, springs, lubricants, friction of bearings, static and dynamic balance, tests on cutting tools, tests on aeroplane models, etc. However, there are four chapters, out of twenty-two, which will chiefly interest the structural engineer. These chapters deal with tests on reinforced concrete slabs, beams, columns, etc., and describe the apparatus and methods used. These tests are taken from those made either in England or the U.S.A., and it may be that this part of the work would have been improved had reference been made to Continental experiments. Chapter XVIII is of particular interest in that tests on actual structures, such as railway bridges, retaining walls, etc., are dealt with, giving excellent reproductions of the autographic diagrams taken. The volume contains (amongst its 446 pages) 313 figures, of which sixty-eight are excellent half-tone blocks ; forty-seven tables, and at the end of every chapter numerous references are given. These, together with clear, short discussions of the underlying principles involved in the tests, make the work one which can be highly recommended to the engineering profession as a whole.

H. E. L. M.

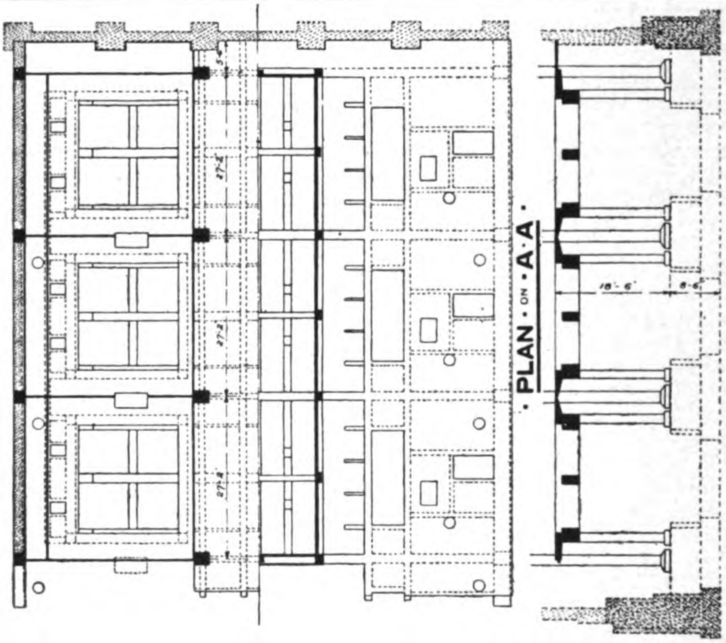


FIG. 2.

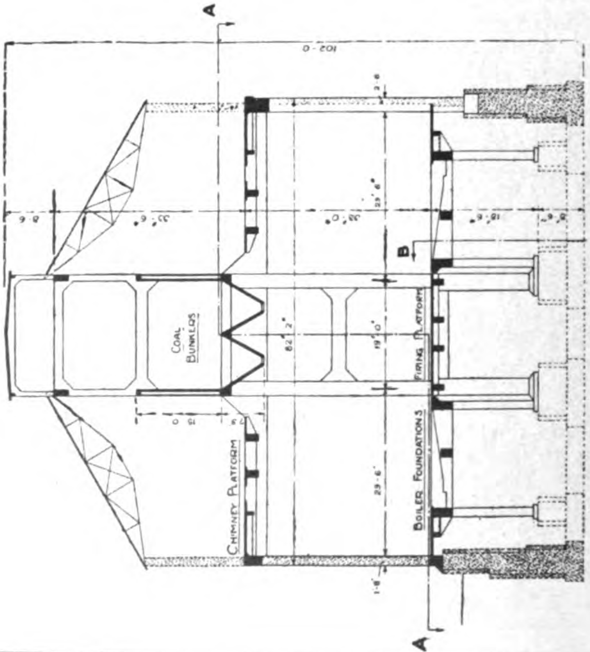


FIG. 1.

REINFORCED CONCRETE POWER HOUSE, BARGOED. (*See p.* 183.)

REINFORCED CONCRETE BUILDINGS IN SOUTH WALES.

THE buildings illustrated herewith have recently been completed for Messrs. The Powell Duffryn Steam Coal Company in connection with their Bargoed Colliery. The complete electrification scheme included two large electric generating stations, one of which (that illustrated) is built of reinforced concrete. The structures carried out in reinforced concrete comprise (1) a building for the steam-generating plant, (2) six separate boiler foundations, and (3) a transformer house.

The building to accommodate the steam-generating plant (as shown in *Figs.* 1, 2 and 3) has sixteen columns, in four rows, carried up from the foundations to carry a firing platform between the two interior columns at a height of 27 ft., and two chimney platforms in the outer bays at a height of 60 ft. The two interior rows of columns have been continued upwards to carry a line of coal-bunkers having a depth of 22 ft., a gantry for an overhead coal-conveyor, and the roof, the top of which is at a height of 102 ft. above the foundation level. A series of six Babcock & Wilcox water-tube boilers has been erected, in two lines, on each side of the firing platform, while the smoke-chambers and fans for forced draught are carried on the chimney platforms. The elevated coal-bunkers have a capacity of 700 tons of coal, which travels from the bunkers to the stokers by gravitation.

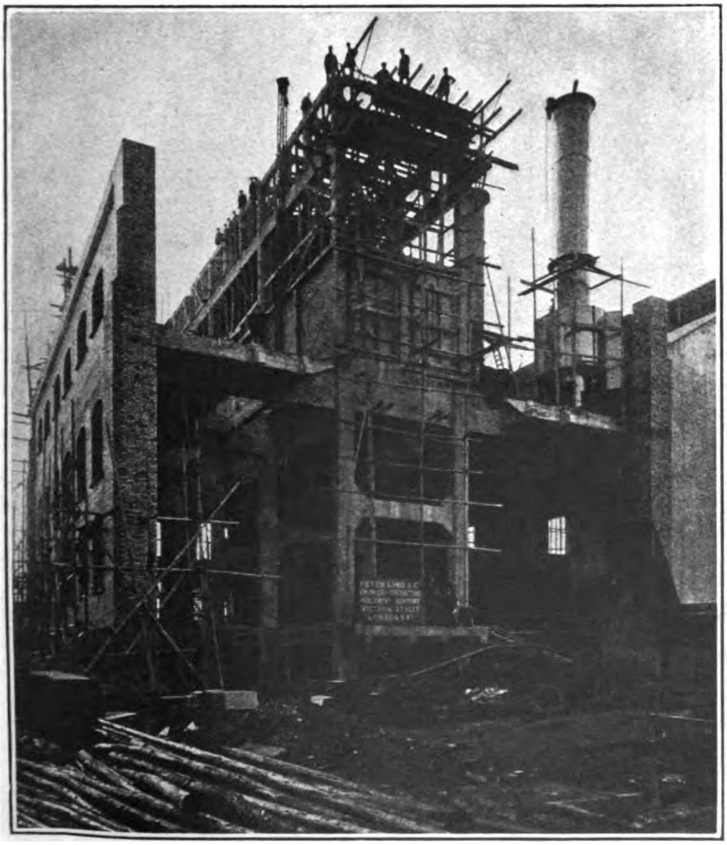


FIG. 3.—REINFORCED CONCRETE POWER HOUSE, BARGOED: DURING CONSTRUCTION.

As the boiler foundations are exposed to great heat, it was decided not to build them in with the main structure. Each foundation, therefore, is self-contained, and consists of four columns which at the level of the firing platform carry a system of beams and slabs shaped so as to fit the stokers. The usual boiler stanchions and brick enclosure are erected on the top of the beams, and to the underside of the beams are bolted the steel ash-hoppers from whence the ashes fall into bogies for removal.

The transformer house is about 70 ft. long by 50 ft. wide by 38 ft. high from ground floor to roof. A compartment, 9 ft. wide by 18 ft. 6 in. wide, along one side of the building provides accommodation for seven transformers at ground level. The switchgear is on the first floor, which is heavily loaded by the stone cubicles for the switches; the floor girders have a span of 31 ft.

These buildings were erected under the supervision of Mr. Ivor Williams, Agent of the Powell Duffryn Company, who was then the Company's Engineer for the Rhymney Valley.

A reinforced concrete provender plant which contains eight corn-bins (each 25 ft. deep) and two reinforced concrete floors underneath for cutting and bagging the provender, has just been completed for the Powell Duffryn Company at Aberaman.

The whole of the reinforced concrete in these buildings was designed by Messrs. Peter Lind & Co., of Westminster, who were also the contractors for the execution of the work.

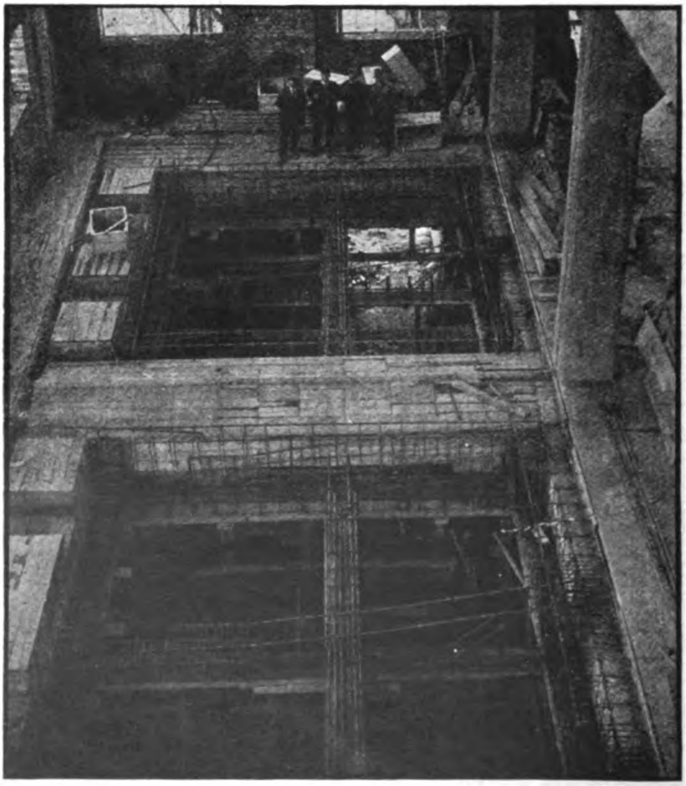
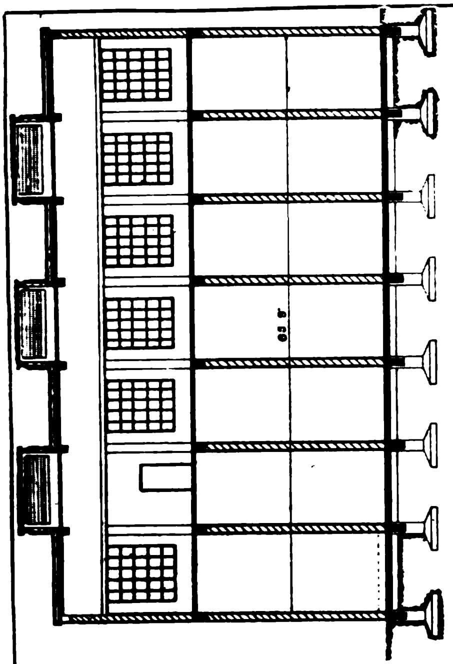
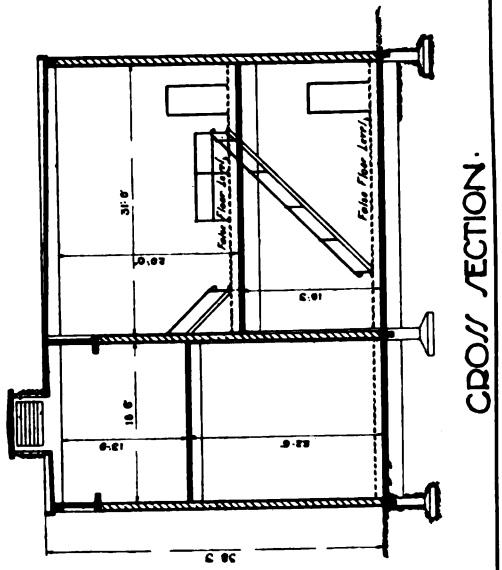


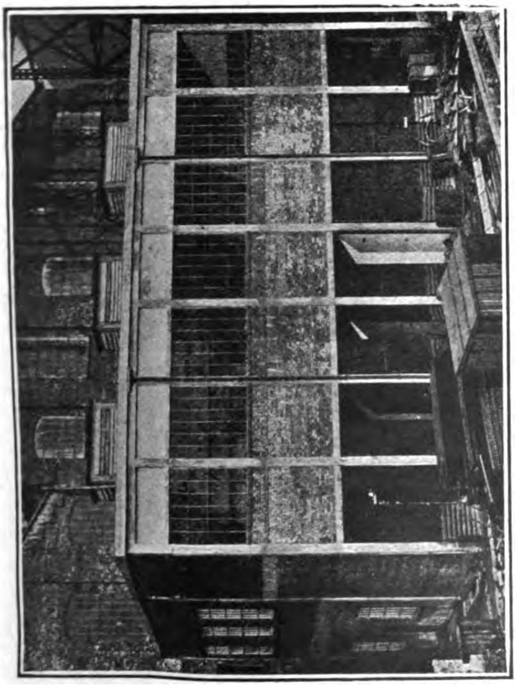
Fig. 4.—Reinforced Concrete Power House, Bargoed: Boiler Foundations.



LONGITUDINAL SECTION



CROSS SECTION.



Reinforced Concrete Transformer House at Bargoed.

(*See p.* 183.)

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—Ed.

REINFORCED CONCRETE CONSTRUCTION.

Questions.—(1) A coal bin is to be of the form shown in *Fig.* 1. How is the pressure on the sloping bottom arrived at? Is it got by resolving the pressure *j*, *k*, *f*, *c* combined with W normally to DC, or by simply resolving W?

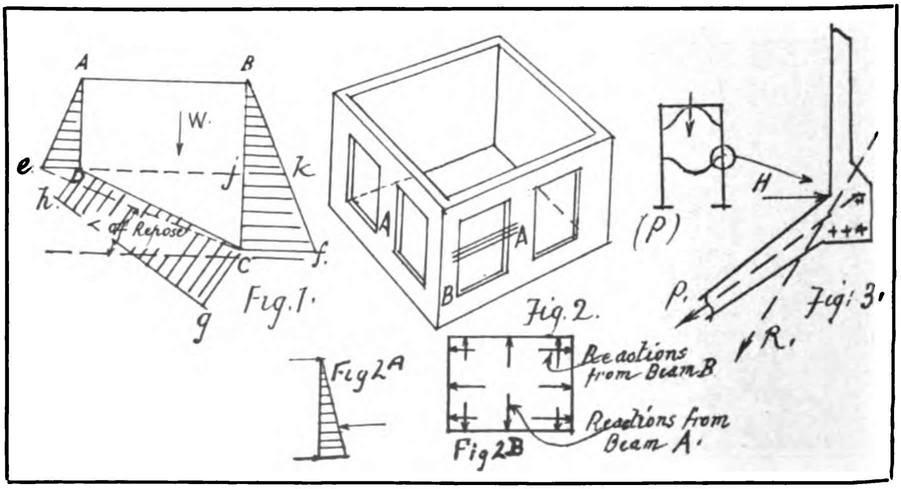
(2) A concrete water tank is too large for the sides to be treated as slabs of span equal to the length and breadth of tank, and a framework of beams is proposed as in *Fig.* 2. If the side slabs are reinforced to span horizontally the intermediate beams (A) are loaded as in *Fig.* 2 A, which I presume are designed for the condition of bottom end fixed and top freely supported. The corner beams (B) are loaded similarly, but in two directions at right angles to each other. How are they treated? If the top and bottom beams are taken as continuous the loading seems to be as in *Fig.* 2 B, and I am doubtful as to the treatment for bending moments. Perhaps you would outline the treatment generally for me.

(3) *Fig.* 3 shows the beam at the junction of the vertical side and sloping bottom of a hopper. This beam seems to take the horizontal pressure (H) transmitted from the side slabs and the inclined pull from the sloping bottom, so that it seems to me the bending would not wholly be in a vertical plane but somewhere in the direction R. Yet I gather from the illustrations of reinforcement in some hoppers that the vertical loading only has been considered. How should they be treated?

"Student."

Answers.—(1) The pressure may be calculated, as suggested, by resolving the lateral pressure *j*, *k*, *f*, *c*, combined with the vertical pressure W, normally to DC.

(2) The conditions of fixity of the intermediate beams A cannot be determined without knowing more than is stated about the bottom. Unless the bottom is monolithic with a beam in the slab, or a very thick slab, it would be well to assume only partial fixity at the bottom. The corner beams B are for appearance only, since the corners are held against outward movement by the



reinforcement to the slabs. The top beams may be treated as loaded at midspan and fixed at the corners. The bottom beam is for appearance only, as the bottom slab would take the reactions direct.

(3) Generally speaking, the horizontal pressure H combined with the inclined tension P will leave approximately a vertical resultant. The forces P and H adjust themselves approximately to this condition.

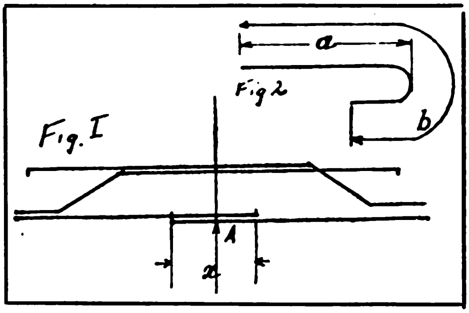
REINFORCEMENT.

QUESTIONS.—(1) In the case of a reinforced concrete slab which is continuous over the support (A), in which one-half of the tensile reinforcement in the centre of the slab is bent up over the support, thus giving 50 per cent. of the area of the steel in compression at the support, what amount of bond length (x) should be given to these rods in the compression side of the slab ?

(2) The amount of bond length is usually given as so many diameters of bar, thus forty diameters for a bar with hooked ends. Is the bond length measured by the straight length (a) to the end of the hook, or by the curved length (b) round the hook. A. S.

ANSWERS.—(1) This depends largely on the percentage of steel at the support. Where this is high, and the compression steel is fully required, the bond length should be about 30 diameters on the compression side. Where the percentage is low, a lesser length is often used.

(2) The bond length is generally measured as the straight length (a), but the hook has some value and may usually be considered as equivalent to a bond length of about 10 diameters.



REINFORCED CONCRETE CALCULATIONS.

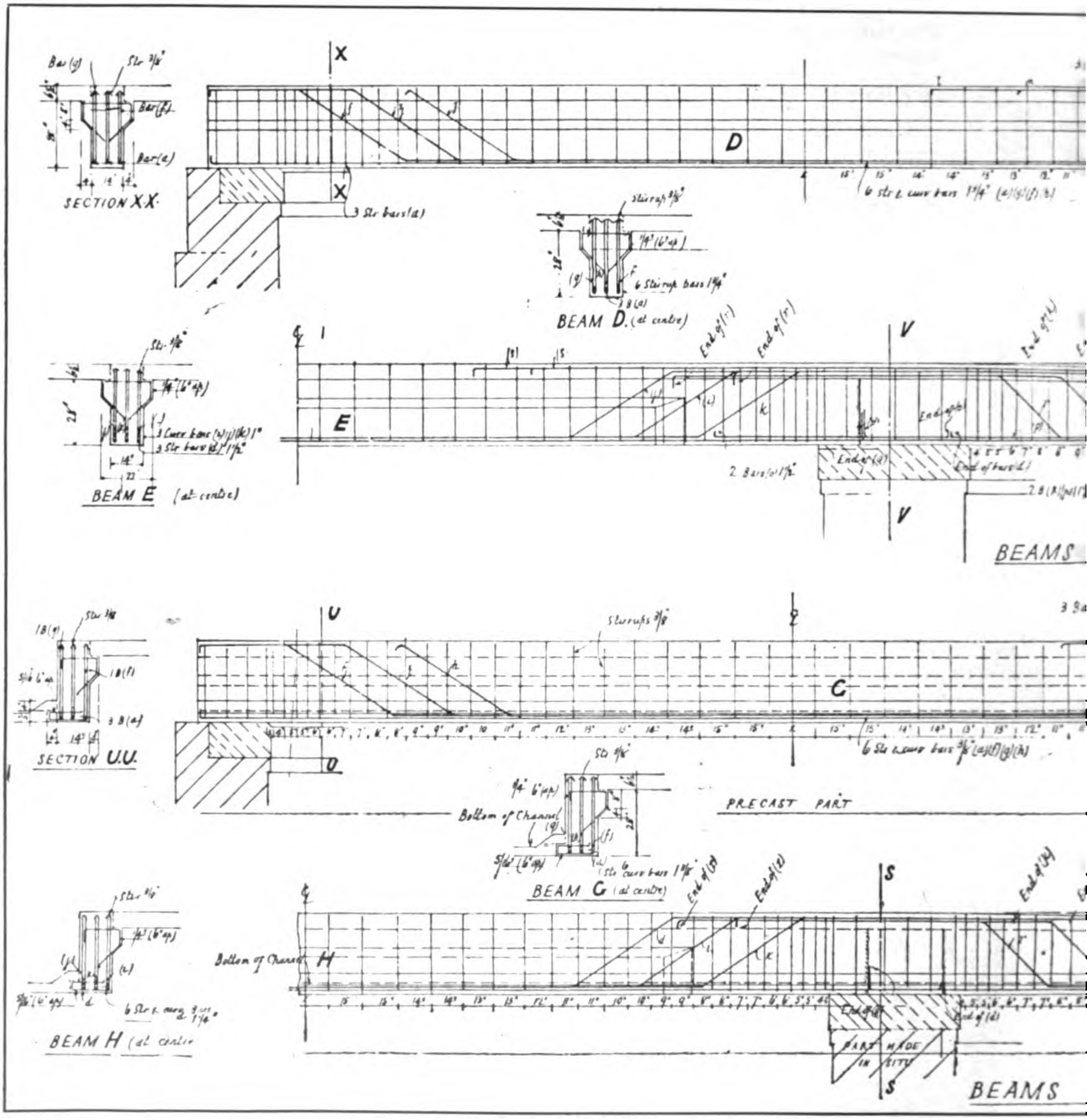
QUESTION.—Having had occasion to use those books on reinforced concrete which I possess, I have been struck with what seems to be a common failing. Complicated formulæ are given for designing such work as columns and arches, and a fair amount of labour is involved in using them. When it is considered that the factors in the formulæ are assumed values, and not accurate to within 10 per cent. or 15 per cent., it is apparent that the formulæ might be replaced by simpler approximations.

As an example, in the formula at the foot of page 103 of Marsh and Dunn's "Manual" the $(m - 1)\,Is$ and the $(m - 1)\,\omega$ factors only affect the result to 2 per cent. or 3 per cent. It is based on

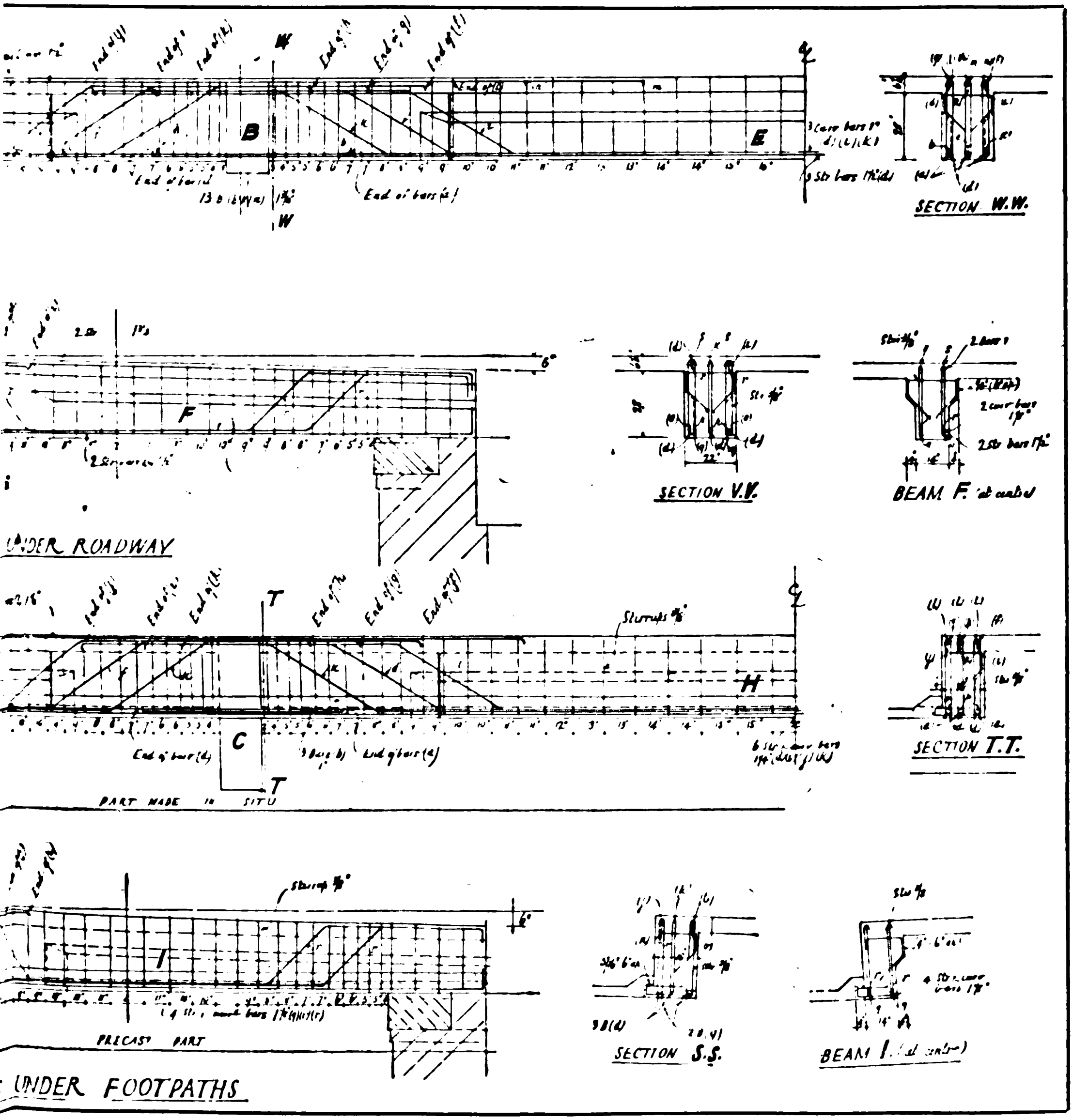
(i) An "assumption" on line 10.
(ii) A "safe" resistance on line 17.
(iii) A "co-efficient" on line 19.
(iv) An "approximation" on line 23 of $m = 15$.
(v) A measurement from a diagram on line 27.
(vi) A second "assumption" on line 29.
(vii) And a third on line 30.

Some of these (such as "C") may be 5 per cent. wrong or more, and therefore formula (3) cannot be trustworthy even to this extent. As an example, the author, on page 105, works out a value to 34 lb. in 78,000. If the factors mentioned above are omitted from formula (3) the answer is some 76,000 lb., and surely as accurate as one is justified in claiming. Messrs. Marsh and Dunn's book is one of the most useful I have, but would, I think, be improved if certain equations were expanded and the first, or at most the first and second terms, were taken as useful approximations. A.D.

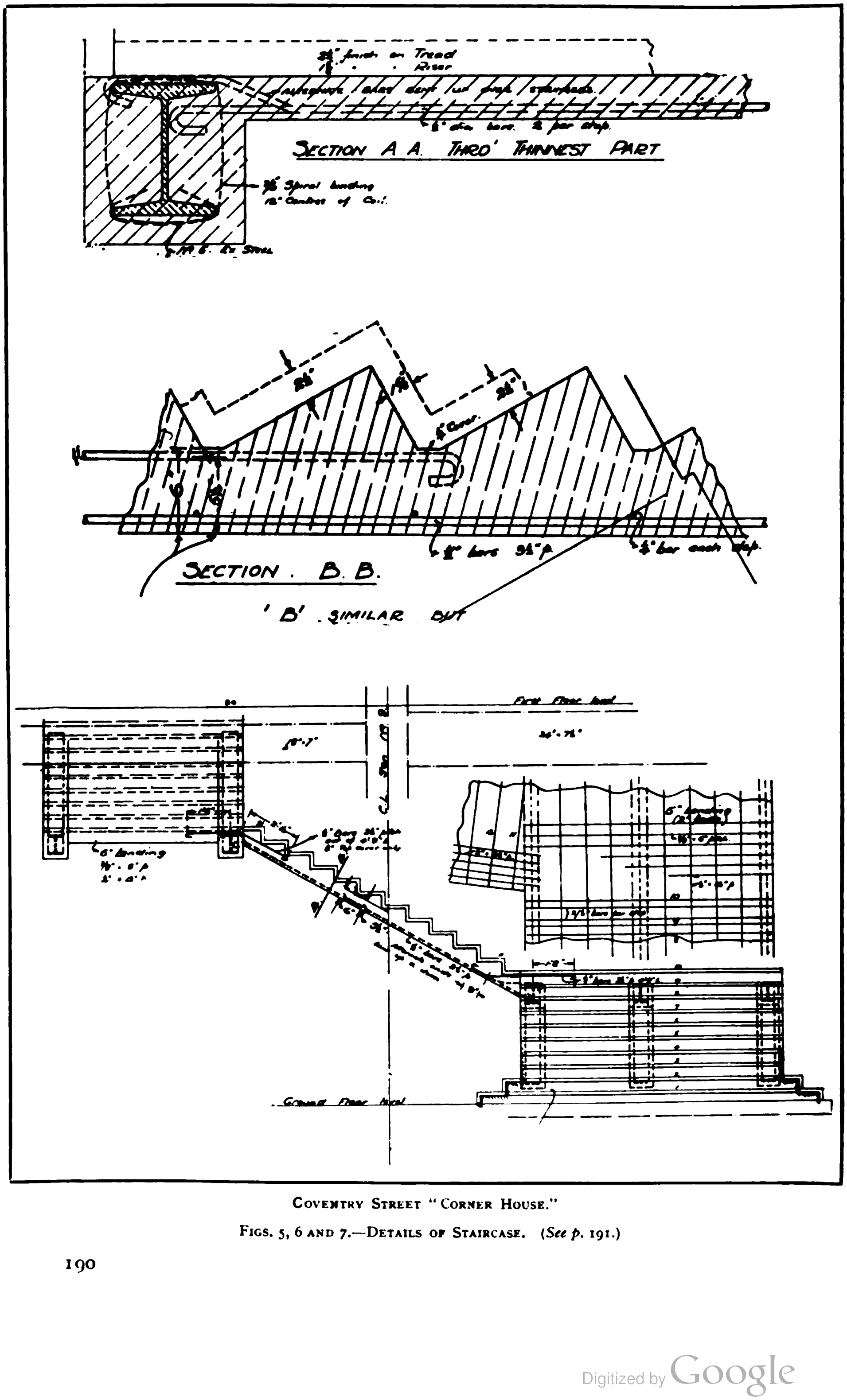
ANSWER.—It is quite true that accuracy exceeding 5 per cent. or 10 per cent. is of doubtful value in reinforced concrete calculations. But in actual design nearly everything is referred to curves, and it is as easy to work from an accurate curve as from a less accurate one. This being so, there are many obvious advantages in keeping the basis formulæ for such curves as accurate as possible.



COMBINATION PRE-CAST AND "IN SITU" BRIDGE,



UNNERSBURY: SECONDARY BEAMS. (*See p.* 169.)



Coventry Street "Corner House."

Figs. 5, 6 and 7.—Details of Staircase. (*See p.* 191.)

OVENTRY STREET "CORNER HOUSE" EXTENSION.

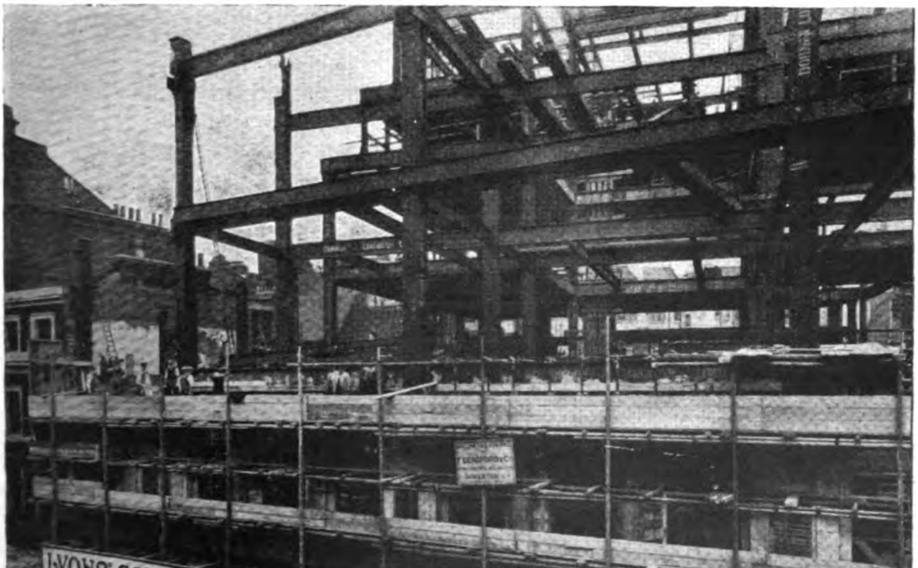
view of the increasing popularity of heir well-known " Corner House " restaurants, Messrs. J. Lyons & Co., Ltd., re carrying out a large extension of their stablishment in Coventry Street, W.C., nown as the " Coventry Street Corner House," and the work is now nearing ompletion. The extension, which adoins the existing building, has a frontage to Coventry Street of 118 ft., and an average depth of 160 ft. The height from the pavement level is 112 ft. The building comprises seven floors above ground, a basement, and a sub-basement, and has a total floor area of about 10,500 yds. super. The basement and sub-basement are to be used for kitchens and service, while in the two top floors machinery will be installed for the manufacture of chocolate; the intermediate floors will be used as restaurants.

The structure is steel frame with hollow tile floors, and main staircases in reinforced concrete.

The hollow tile floors, which vary in thickness from 7 in. to 9 in., were designed for live loads of 1 cwt. and 2 cwt. per ft. super, and when tested under working conditions successfully withstood loads of 7 cwt. per ft. super. The floor spans vary in width from 8 ft. to 17 ft., and in length up to 38 ft. across the bays. The surface of the floors is to be finished in mosaic, wood blocks, and tiles, the service-room floors all being finished in tile.

The steelwork is cased throughout in concrete to comply with the London County Council Regulations. The steps and landings and all the staircases are in reinforced concrete, carried on steel stringers cased in concrete.

A somewhat unusual form of construction has been adopted in connection with the Mansard roofs in order to overcome the difficulty of fixing battens to take the tiles. As shown in *Fig.* 4 the method adopted is to introduce breeze concrete



Coventry Street " Corner House ": View during Construction.

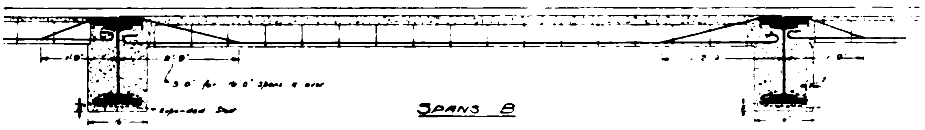


FIG. 1.

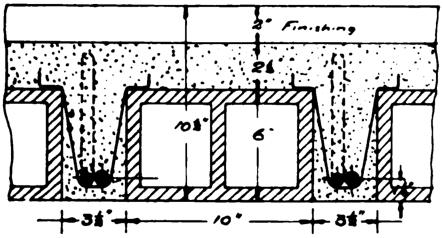
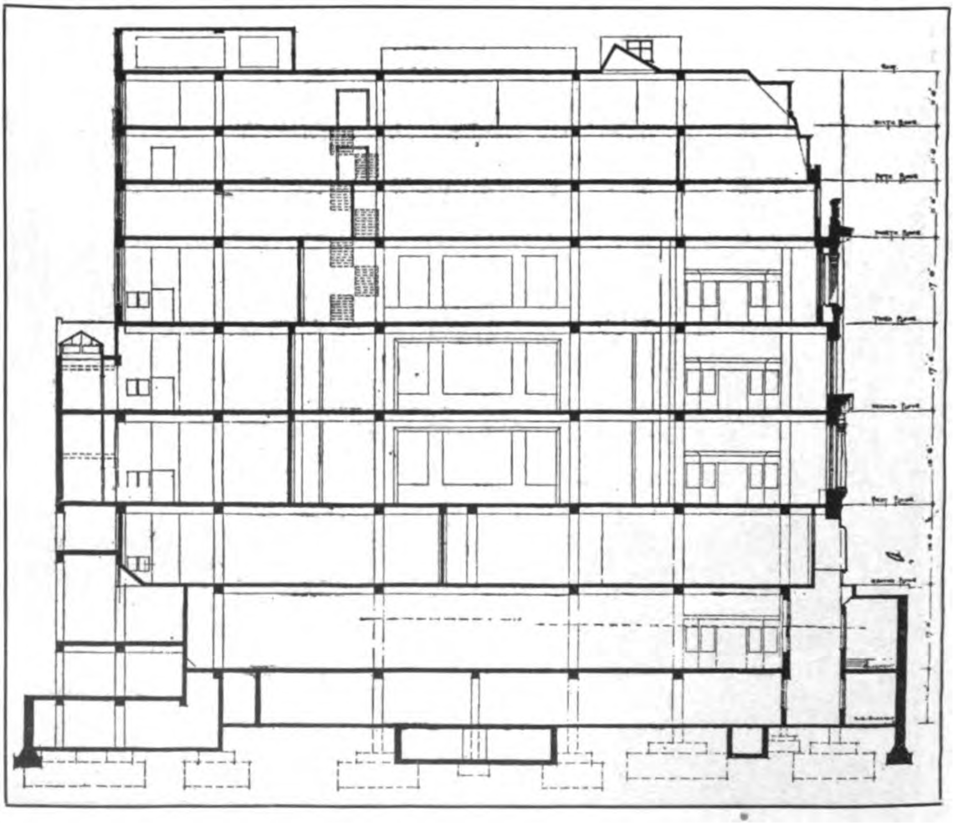


FIG. 2.—DETAIL OF HOLLOW BLOCK FLOOR, 15 FT. SPAN.

slabs on top of the concrete roof slabs so that the battens can be nailed on to them.

The construction of the floors and staircases is illustrated in *Figs.* 1, 2, 3, 5, 6 and 7.

The architect for the building is Mr. F. J. Wills, of 62, Oxford St., W., and the whole of the floors, roof, staircases, and concrete casing to steelwork is being carried out by Messrs. F. Bradford & Co., of Coopersale Road, Homerton, E.9, who are now specialising in this type of hollow floor construction.



COVENTRY STREET "CORNER HOUSE": LONGITUDINAL SECTION THROUGH BUILDING.

THE INSTITUTION OF STRUCTURAL ENGINEERS.

ANNUAL DINNER.

If the success of the first annual dinner under its present title is any indication, as we hope it is, the Institution of Structural Engineers has a very promising future before it. The dinner, at which over 250 members and friends were present, was held at the Savoy Hotel last month under the chairmanship of Mr. E. Fiander Etchells, the President, and, as was the case last year, arrangements had been made for the gathering to use the ball-room later in the evening. The after-dinner speeches were interspersed with an excellent musical programme.

The loyal toasts having been honoured, Mr. Ewart S. Andrews introduced what he hoped might be a regular feature at the dinners of the Institution in the future. His toast was "To the Pious Memory of a Great Structural Engineer"—John Smeaton, the engineer of the Eddystone Lighthouse, in regard to whose attainments in his profession he quoted the opinion of George Stephenson that he was "the

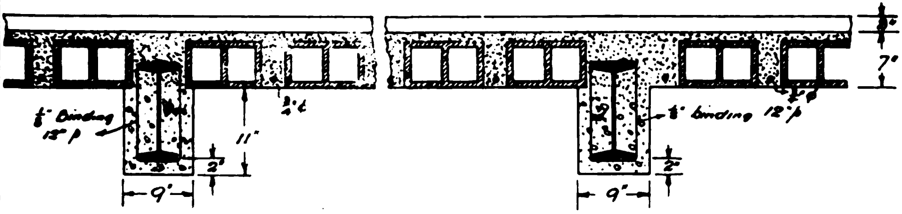


Fig. 3.—Section of Floor up to 12 ft. Span.

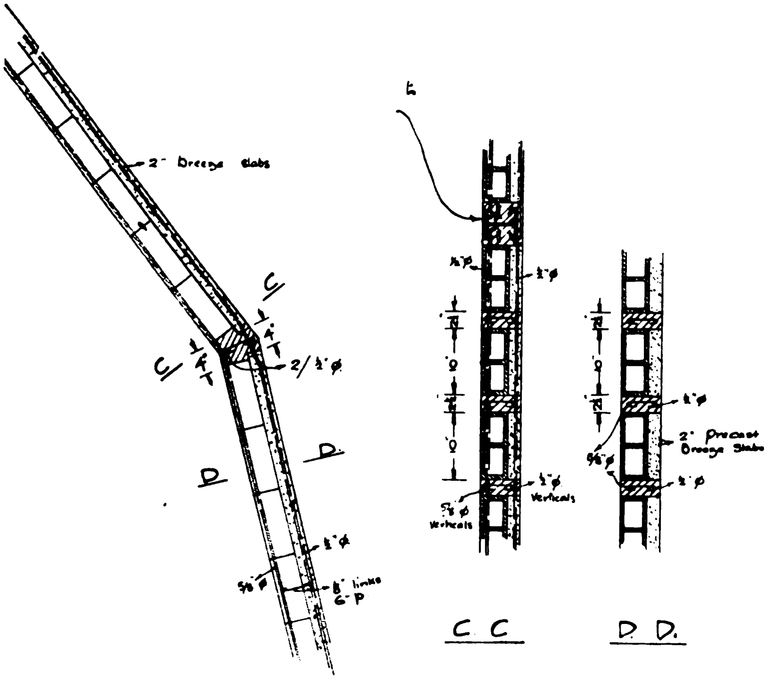


Fig. 4.—Detail of Mansard Slopes.

Coventry Street "Corner House" Extension. (*See p.* 191.)

most eminent engineer this country has ever produced." Mr. Andrews said that in the rush and worry of modern life, and in the dazzle of recent accomplishments, present-day engineers were apt to forget the debt of gratitude they owed to the past masters of their profession. He thought it would be a very good thing, therefore, to give a few minutes to honour the memory of one of the great men who had gone before them. Smeaton's best known work had stood for over 160 years as a monument to his skill, as a beacon of welcome to those returning from abroad, and as an emblem of the strength and reliability of character which they were proud to believe were characteristic of the British nation. The speaker concluded a sincere tribute with a brief outline of Smeaton's life-history, and the toast was drunk in silence.

Sir Charles T. Ruthen, O.B.E., proposed "His Majesty's Services," and in responding Major-General Sir W. S. Brancker (Director of Civil Aviation) made some interesting forecasts as to the development of air transport. After pointing out that the fact that no passenger was killed (and only one injured) in the cross-Channel aeroplanes and "joy-riding" trips by air during the past year was sufficient evidence of the safety of that means of transit, he predicted that within five or six years commercial air transport would be a paying proposition. He pointed out that when it came about it would bring in its train a considerable volume of work for structural engineers in connection with the aerodromes and landing places. He mentioned schemes which had been put forward for covering Hyde Park, part of the river, or one of the large railway stations for the purpose, but those of his hearers who had the amenities of London at heart no doubt welcomed his assurance that the idea had been turned down by the Government. Flying at an altitude of 30,000 ft., flights at 300 miles an hour and trips to America in one day were among his predictions of what would be common feats within a very few years.

Lord Riddell proposed "The Institution," and in the course of an entertaining speech pointed out the importance of the æsthetic side of engineering, and urged those responsible for the construction of buildings to see that the amenities of the towns and countryside were not spoiled by their erections.

The President responded in characteristic manner. He commenced by giving "translations" of a few Latin quotations, including: *Tertium quid*—a legal term meaning six shillings and eightpence; *Sub judice*—the bench on which judges sit; *Magna est veritas et praevalebit*—Great is truth, it shall prevail a bit; *Si monumentum requiris, circumspice*—If a monument you want, be circumspect. Then followed the reading of a manuscript alleged to have been found in the tomb of Tutankhamen—the President apologised that it was "written in hieroglyphics and difficult to decipher," although on close inspection the writing has a remarkable resemblance to his own!

Major James Petrie, O.B.E., the Rt. Hon. Sir Charles Darling, and the Rev. F. H. Gillingham also spoke.

ARCHITECT AND ENGINEER.

We give below a report of the discussion on the paper on "The Architect and Engineer" read at a meeting of the Institution of Structural Engineers on January 25 by Mr. W. J. H. Leverton. An abstract of the paper was given in our last issue.

Mr. William Woodward, F.R.I.B.A., said the architect should be a man of art and also have a considerable knowledge of construction in regard to the structure he was about to put up. At the same time, there were three people who should come into the construction of any large building, namely, the architect, the engineer, and the quantity surveyor. They were all distinct branches, and should be left to themselves, under the supreme control, however, of the architect. In connection with the restoration and repair of ancient monuments by the Office of Works, he said Tintern Abbey was an example of the result of that policy. It had been restored, but in what a fashion! No attempts had been made to restore the old stone-work, but steel girders had been employed. The roof of Westminster Hall had also been repaired with steel-work. It was true that not a bit of the steel-work could be seen, but he did not think that was the manner in which to restore and repair old buildings, and he did not think such methods would have been adopted if the work had been entrusted to architects and not to engineers.

Mr. H. Kempton Dyson said he differed from the author in saying that the engineer was purely utilitarian, because that seemed to imply that the engineer could not produce a good-looking building. He had found that when working with a new material like reinforced concrete it was necessary to get used to the appearance of the material in order to deal accurately with it as regarded proportions. The architect, by reason of his training, could produce a building in the usual materials which was both pleasing to the eye and sound engineering, but when it came to a new material the architect needed practice to get his eye accustomed to it. In the case of most structures he felt that the engineer should be consulted by the architect at a very early stage in the design, and not be called in when everything had been settled. In the case of bridges he felt the engineer should be in complete control, and not the architect. A consulting engineer should be called in more often than was the case, and the client should be made to appreciate that in most cases this meant economy, and the client should pay for the services of the engineer.

Mr. H. J. Deane said he had known cases in which the engineer who had assisted the architect in preparing the steel-work designs had been told he would be paid by the contractor. That was radically wrong. The architect should include in his fees sufficient to pay for the assistance of the engineer or an arrangement should be made whereby the engineer was paid separately by the client.

Mr. P. L. Wilson said he thought the architect should have the fullest liberty to express his ideas in a design and then get the engineer to carry out those ideas so far as the structural part was concerned. It should not be necessary to curb the architect in his ideas, except where that was necessary for reasons of expense.

Mr. A. S. Spencer said he recommended all architects to employ a consulting engineer in the design of steel-work. As a representative of a firm which carried out steel construction, he knew the value of competitive designing. Where a consulting engineer was not employed and the firm was asked to get out the design things were often done which should not be done owing to the stress of competition in such circumstances, and architects should lean on the consulting engineer in that respect more than they did at present. He knew an instance in which some twenty different firms prepared competitive schemes, and each must have spent £150 in so doing. Naturally, only one firm could get the order; but the employment of a consulting engineer to prepare the scheme and get tenders would have eliminated a great deal of wasted effort and expense.

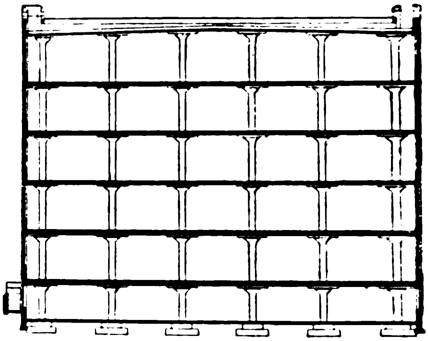
Mr. G. B. R. Pimm, speaking with regard to the remark that there had been cases in which the engineer had to look to the contractor for payment, said that was almost invariably done. The result was that the client often paid twice over for the same thing, and the architect took credit for a design which was prepared by some one else.

A FACTORY BUILDING WITH MAXIMUM WINDOW WALL SURFACE.

In the new building added to the plant of Messrs. James Lees & Sons Company, at Bridgeport, Pa., the walls are almost entirely of glass. This new building, which is an addition to the present plant, will be five stories and basement in height; 121 ft. wide by 253 ft. long, with a total floor area of about 180,000 sq. ft., and will be used for the manufacture of worsted yarns. It is of reinforced concrete flat slab construction, the top wearing surface of the floor being of maple. The exterior walls will be of brick with terra-cotta ornament.

The unique feature in the construction consists of the omission of exterior wall columns, permitting, if desired, an unbroken line of light around the entire building except where broken by stairways or other features.

Apart from the centre and corner pylons, which have been added to the façades to give an architectural effect, there is a total absence of pilasters, thus permitting the steel windows to be made continuous. This results in admitting an unusual amount of light to flood the interior of the building, adapting it to the purposes of manufacturing which in this case required a building 120 ft. wide without an excessive height.

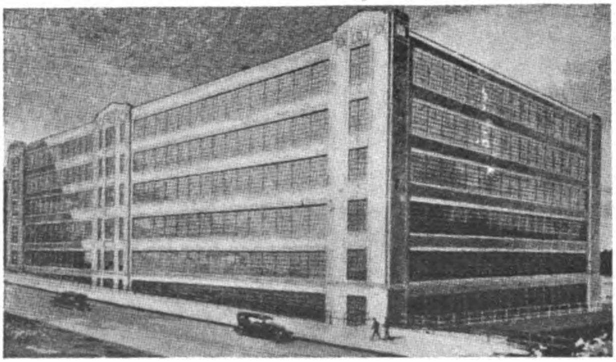


Longitudinal Section, showing Cantilever Construction of Floors.

This unbroken extent of window space was obtained by making use of the principle of the cantilever. The first row of interior columns is placed 5 ft. back from the wall, the concrete floor slabs extending beyond the columns to the wall line, the structural walls under the windows being built directly upon the projecting floor slab. This method gives not only a maximum of daylight, but by the elimination of the wall columns the concrete work is simplified, and, together with the economy obtained by making use of the cantilever principle, results in a considerable saving in cost in the construction of every part of the building.

Another advantage resulting from the omission of the wall columns with their projections on the inside of the building is that the installation of work-benches and other equipment, as well as the radiation for the heating system, is greatly facilitated.

The Ballinger Company, Architects and Engineers, Philadelphia and New York, designed the building and has charge of construction.—*The American Architect.*



Factory Building with Maximum Window Surface.

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	6
Clean Shingle, ¾ in. mesh	,,	14	6
,, ,, ½ in. mesh	,,	15	6
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Portland Cement	10s. per ton extra.		

Boarding for Shuttering—		Sawn.	Wrot.
1 in.	per square	23 6	27 6
1¼ in.	,,	29 6	33 6
1½ in.	,,	35 6	41 0

Sawn Timber for Struts and Supports—	
3 in. by 4 in.	from £22 10s. per standard.
3 in. by 6 in. and 3 in. by 7 in.	,, £23 ,, ,,

Mild Steel Rods for Reinforcement—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	14	6
9/16 in. to ½ in. Rounds	,,	14	6
⅜ in. Rounds.	,,	15	6
¼ in. Rounds.	,,	17	6

MATERIAL AND LABOUR INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

Portland Cement Concrete well mixed with Clean Water in proportion.

1 : 2 : 4—				s.	d.
Do.	do.	in foundation	per yard cube	45	0
Do.	do.	in columns	,, ,,	50	0
Do.	do.	in beams.	,, ,,	50	0
Do.	do.	in floor slabs 4 in. thick	per yard super	5	3
Do.	do.	in floor slabs 6 in. thick	,, ,,	7	9
Do.	do.	in floor slabs 9 in. thick	,, ,,	11	3
Do.	do.	in walls 6 in. thick	,, ,,	8	0

(*Add for hoisting 3s. 6d. per yard cube above ground-floor level.*)

Steel Reinforcement, including Cutting, Bending and putting into Position and securing with Stout Binding Wire—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	27	6
,, 9/16 in. to ½ in.	,,	25	6
,, ⅝ in. to 2⅞ in.	,,	24	6

Shuttering—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	60	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	50	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11
Do. do. as last in narrow widths	,, ,,	1	1
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1
Do. do. as last in narrow widths	,, ,,	1	3
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

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THE HUB OF THE CONCRETE WORLD

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DURING the past 16 YEARS "WINGET" machines have been sold and despatched to the following widely scattered parts of the world:—

BOMBAY	NEW ZEALAND
CAPE TOWN	BELGIUM
SINGAPORE	THE BELGIAN CONGO
SHANGHAI	PORTUGAL
TOKIO	THE GAMBIA.

and many parts of England, Scotland and Ireland.

DURING the past 16 YEARS countless housing schemes at home and abroad have been built with "WINGET" Concrete Blocks. "WINGET" Machines have been bought and used by all the British and most of the Foreign Government Departments, as well as by the leading corporations, gas and railway companies, and contractors famous all over the globe.

THESE are the reasons why Winget—a household word in contracting circles—remains the premier firm of concrete and kindred machinery *manufacturers*. British Capital, British Brains, British Labour—not merely naturalised but British through and through, and therefore thorough.

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MEMORANDA.

Professional Announcement.

Mr. Alfred S. Grunspan, B.Sc. (Ldn.), A.M.Inst.C.E., A.C.G.I., Consulting Engineer, has removed to " Abbey House," 8, Victoria Street, Westminster, S.W.1. Telephone: Victoria 4746. Telegrams: " Grunspan, Vic., London."

Concrete Sewer Tubes in South Africa.

In a paper to be brought forward for discussion before the Institution of Civil Engineers, Mr. D. E. Lloyd-Davies, M.Inst.C.E., states that in connection with the main drainage of the southern suburbs of Capetown the sewer tubes of from 18 in. to 40 in. diameter are all of granolithic concrete with a covering of ordinary concrete, and culverts of over 40 in. diameter are constructed of concrete poured *in situ*.

Concrete Railway Sleepers.

A very thorough test of a new type of concrete railway sleeper, consisting of a pair of concrete blocks connected by a tie-bar, patented by Mr. D. H. Stent, a consulting engineer of Delhi, has been made by the North-Western Railway of India on a stretch of some 50 miles of main line near Delhi. The Company are so satisfied with the behaviour of this sleeper that they propose to lay a further stretch of 70 miles. This, together with the reference in our December issue to the fact that the East Indian Railway and the Bengal and Nagpur Railway are about to manufacture concrete sleepers on a large scale, shows that the value of this type of sleeper is fully realised in India.

Concrete Pipes in Irrigation Works.

A total of 1,390,700 ft. of concrete pipes is being used in the irrigation of an area of 9,000 acres at Loveday, in the Murray Valley, South Australia. The pipes, which are made by the Hume process, vary in diameter from 6 in. to 54 in., and have a total weight of 35,000 tons.

Photo: Campbell Gray.

Concrete Floors at the British Empire Exhibition.

[This view shows the concrete floors, extending over twenty acres, which have recently been completed at the British Empire Exhibition, Wembley. The floors are 4½ in. thick. The contractors are Sir Robert McAlpine & Sons, and the whole of the concrete was mixed in four " Rex " mixers, supplied by Messrs. Gaston, Ltd., of Acton Vale.]

PROSPECTIVE NEW CONCRETE WORK.

Ashford.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the U.D.C. for sanction to borrow £11,975 for sewerage and sewage disposal works.

Barmouth.—*Bridges.*—The directors of the Great Western Railway Co. have authorised the reconstruction of the large bridges over the River Wnion between Barmouth and Dolgelly.

Bedhampton.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the Havant R.D.C. for sanction to borrow £13,860 for sewerage and sewage disposal works in Bedhampton.

Birkenhead.—*Dock.*—At a meeting of the Mersey Docks and Harbour Board, a scheme was submitted involving the expenditure of £230,000 for the improvement of the Alfred Dock.

Bollington.—*Road.*—The U.D.C. proposes to construct a new road connecting Wellington Road with the Stockport and Macclesfield Road, Adlington, at a cost of £20,070.

Bournemouth.—*Groynes, etc.*—The Corporation has decided to apply for sanction to borrow £53,500 for sea-wall groynes and a retaining wall.

Bury St. Edmunds.—*Swimming Pool.*—The Ministry of Health has held an inquiry into an application by the T.C. for sanction to borrow £2,688 for the construction of an open-air swimming pool.

Cambridge.—*Open-air Bath.*—The T.C. has decided to construct an open-air bath at Jesus Green, at an estimated cost of £2,100.

Cardiff.—*Road Works.*—The Corporation proposes spending £125,000 on the reconstruction and improvement of a number of city roads.

Castletown.—*Road.*—The Sunderland R.D.C. has under consideration a proposal to construct a new road from Castletown to North Hylton.

Chadderton.—*Road.*—The Manchester City Council has approved a scheme for a new road from Moston Lane to Greengate, and from Moston Lane to Bower Street, Chadderton. The total cost is estimated at £273,000.

Dewsbury.—*Road.*—The Corporation proposes to construct a new road from the Midland Railway Station to join the Huddersfield Road. The estimated cost is £102,000.

Duntocher.—*Boulevard.*—The Glasgow Corporation has approved the construction of a boulevard from Anniesland Cross to Duntocher, at an estimated cost of £500,000.

Faversham.—*Road.*—Representatives of various Associations in Broadstairs, Margate, Ramsgate and Westgate are to meet to consider the question of constructing a coastal road from Faversham to Thanet.

Gosport.—*Road Work, etc.*—The T.C. has received sanction to borrow £8,000 for concreting various roads, and £5,750 for the construction of a swimming bath at the "Wides." An application has also been made for a loan of £7,000 for a new bridge at Alver Road.

Great Yarmouth.—*River Wall.*—The Corporation has decided that Breydon Hall be faced with concrete, where necessary, at a cost of £1,300.

Hull.—*Bridge.*—The Corporation is considering an estimate of £70,000 for work, which includes the construction of a bridge near Neptune Street Goods Station.

Hull.—*Concrete Flooring.*—The Corporation Parks Committee has approved an estimate of £2,353 for providing a concrete floor for the proposed open-air dancing enclosure in East Park.

Kettering.—*Road.*—The U.D.C. has decided to proceed with the construction of a new arterial road from Station Road to Northampton Road, at a total estimated cost of £4,300.

Kingston.—*Roads.*—The Surrey C.C. has agreed to co-operate with the Ministry of Transport and the L.C.C. in the construction of two new arterial roads in Surrey. One is known as the Kingston by-pass and the other as the Sutton by-pass, and the total estimated cost is £490,000.

Knuzden.—*Sewage Works.*—The Oswaldtwistle U.D.C. has decided to apply to the Ministry of Health for sanction to borrow £4,366 for carrying out the Knuzden sewage scheme.

Leith.—*Dock.*—The Leith Dock Commission has approved an expenditure, not exceeding £25,000, on the first section of the proposed new quay wall of the outer harbour.

Liverpool.—*Road.*—The Lancashire C.C. has decided to proceed with the construction of the proposed new main road between Liverpool and Manchester at an estimated cost of £3,000,000.

London (Hackney).—*Road Works.*—At a meeting of the B.C., the Works Committee submitted recommendations involving an expenditure of £58,460 for the reconstruction of roads.

London (Tooting).—*Tennis Courts.*—It is stated that hard tennis courts are to be laid on Tooting Common.

Lowestoft.—*Sea Wall, etc.*—The Corporation has received sanction to borrow £96,300 for the construction of the north beach sea wall and the strengthening of the groyne.

Middlesbrough.—*Docks, etc.*—A scheme for the construction of a ferry landing and docks, at an estimated cost of £20,000, has been approved by the Corporation Ferry Committee.

Nelson.—*Bridge.*—The T.C. has agreed to construct a new Carr Road bridge at a total estimated cost of £7,000.

Newburn.—*Road.*—The U.D.C. is considering a proposal to construct a new road from Grange Road, Newburn, to Falconer Terrace, North Wylam. The estimated cost is £45,900.

Oxford.—*Bridge.*—The Berkshire C.C. has agreed to make a contribution of £3,500 towards the cost of rebuilding Seacourt Bridge by the Oxford Corporation. The estimated cost is £15,000.

Pontypridd.—*Bridge.*—The U.D.C. has given instructions for the preparation of plans and estimates of (*a*) carriageway ferro-concrete bridge, and (*b*) ferro-concrete footbridge over the River Taff.

Portsmouth.—*Concrete Work.*—Amongst the work proposed by the Corporation is the widening of the quay in reinforced concrete at the Outer Chamber; the rebuilding of a length of the quay wall in reinforced concrete; the provision of a reinforced concrete jetty, and a flight of reinforced concrete steps.

Preston.—*Open-air Bath.*—The Corporation proposes constructing an open-air bath on a portion of land adjoining Waverley Park.

Ramsgate.—*Sea Wall.*—The Corporation has decided to make application for sanction to borrow £54,000 for the construction of a sea wall and other works.

Scarborough.—*Road.*—The Borough Engineer has received instructions to put in hand the construction of a new road over the Walmsley estate.

Shepsted.—*Sewage Works.*—The U.D.C. has decided to apply to the Ministry of Health for sanction to borrow £8,400 for the construction of sewage works.

Shoreham.—*Groynes.*—The Shoreham and Lancing Sea Defence Commissioners are considering the question of constructing four groynes at an estimated cost of over £4,000.

Southampton.—*Dock Work.*—The Southern Railway proposes constructing a new dock and five jetties at an estimated cost of over £10,000,000.

Stockport.—*Sewage Works.*—The Corporation has received sanction from the Ministry of Health to borrow £180,000 for sewage disposal works at Cheadle Heath.

Thornton (Lancs).—*Sea Wall, etc.*—The U.D.C., in a town-planning scheme, is providing for a sea wall and promenade estimated to cost £94,000; an open-air swimming bath, £20,000; a new road and widening of existing road, £26,000; reinforced concrete culverts with penstocks, and a jetty 400 ft. in length.

Walsall.—*Road.*—The Corporation is advised to proceed with the construction of a ring road at a total estimated cost of £62,309.

Warwick.—*Concrete Wall.*—The T.C. is proposing to construct a river walk with a reinforced concrete retaining wall, and a reinforced concrete footbridge over the river, at an estimated cost of £15,000.

Warminster.—*Swimming Bath.*—To provide work for the unemployed, the U.D.C. propose constructing a concrete swimming bath 70 ft. by 35 ft.

Wednesbury.—*Sewage Works.*—The T.C. has received sanction from the Ministry of Health to borrow £13,500 and £1,500 in connection with the sewage works extensions.

Wellingborough.—*Road.*—The U.D.C. proposes constructing a new road on the outskirts of the town to join the Doddington and Northampton Roads.

Whitehaven.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the R.D.C. for sanction to borrow £1,400 and £3,600 for sewerage and sewage disposal works at Lamplugh and Distington respectively.

Wimbledon.—*Concrete Wall.*—The T.C. is proposing to erect concrete walling in various parts of the borough at an estimated cost of £5,688.

Wolstanton.—*Road.*—The U.D.C. has decided to carry out the construction of a new road from Wolstanton to Chesterton.

Wolverhampton.—*Road.*—The Dudley T.C. has, it is stated, approved the scheme for the proposed new road from Birmingham to Wolverhampton, and has also agreed to contribute £20,000 towards the cost.

Wood Green (Middlesex).—*Culverts, etc.*—Amongst the work to be commenced shortly by the U.D.C. is a reinforced concrete culvert, Moselle Brook, £6,000; a concrete tube culvert, Muswell Stream, £6,000; and a depôt boundary wall, £1,300.

TENDERS ACCEPTED.

Fordyce.—*School Buildings.*—The Banff Education Authority has accepted the tender of Messrs. J. Scott & Son (Aberdeen), Ltd., at £283, for the concrete and asphalte work in connection with the proposed new school at Fordyce.

Hereford.—*Sewage Works.*—The T.C. has accepted the tender of Messrs. W. Bowers & Co., Hereford, at £3,096, for ferro-concrete tanks in connection with the sewage works at Hunderton.

Leeds.—*Roof.*—The T.C. has accepted the tender of Messrs. D. G. Somerville & Co., Manchester, at £4,721, for the construction of a reinforced concrete roof to Dewsbury-road reservoir.

Newcastle-under-Lyme.—*Sedimentation Tanks.*—The Corporation has accepted the tender of Messrs. J. Taylor & Son, Stoke-on-Trent, at £2,760 1*s*. 2*d*., for the construction of two sedimentation tanks in reinforced concrete.

STRAND-ON-THE-GREEN (MIDDLESEX).—*Embankment Walls.*—The Chiswick U.D.C. has accepted the tender of Messrs. Davidson & Sykes, Ltd., London, at £3,080, for the erection of concrete river embankment walls at Strand-on-the-Green.

TUNBRIDGE.—*Bridge.*—The Winchester T.C. has accepted the tender of Mr. J. Douglas, Southampton, at £1,397, for the construction of a new ferro-concrete bridge at Tunbridge.

WEMBLEY.—*Sewer.*—The U.D.C. has accepted the tender of Mr. C. F. Cole, Alperton, at £997, for the construction of about 400 yds. of reinforced concrete pipe surface-water outfall sewer, with manholes, etc.

WIGTON.—*Concrete Pipes.*—The U.D.C. has accepted the tender of the Stanton Ironworks Co., Ltd., Nottingham, at £3,186 5*s*. 8*d*., for the supply of 4,487 yds. of 15 in., and 1,234 yds. of 16 in. concrete pipes.

YEOVIL.—*Dressing Yard, etc.*—Messrs. Ivelnap, Ltd., Yeovil, have accepted the tender of Messrs. R. Butcher & Son, Warminster, at £937, for dressing yard with reinforced concrete pits.

TRADE NOTICES.

The all-concrete arterial roads now being constructed by the Manchester Corporation have been filmed by the Gaumont Company for the Walker-Weston Company, Ltd., of 7, Wormwood-street, E.C.2, whose patent interlocked double-layer reinforcement is being used on the work. The film clearly shows the very difficult nature of the soil, which is in some parts soft peat for a considerable depth, on which the roads are being built, and also demonstrates the manner in which the reinforcement is assembled on the site. It is hoped shortly to have a public exhibition of the film. The Manchester arterial roads were illustrated and described in our issue for November, 1922.

The British Reinforced Concrete Engineering Co., Ltd., has removed its branch office at Leeds, the new address being No. 5, East Parade, Leeds. At this address Mr. McLean Gibson manages the Constructional Department, and Mr. John W. Liversedge the Roads Department.

Mr. Ainslie Reynolds, M.I.Struct.E., has joined the Roads Department of the British Reinforced Concrete Engineering Co., Ltd., and is now at the Company's Offices, No. 4, Iddesleigh House, Caxton Street, Westminster, S.W.1.

Messrs. Manufactured Concrete Works, of 39, Victoria Street, S.W.1, have issued a very complete catalogue of different types of reinforced concrete fencing suitable for all requirements. The various types are all illustrated and dimensioned, and a useful feature is a recommended spacing in each case. Copies of the catalogue will be sent on application.

Woodbridge Road, Ipswich, which forms portion of the main London Yarmouth Road, has been reconstructed in reinforced concrete under the supervision of Mr. S. Little, Engineer and Surveyor to the Ipswich County Borough. B.R.C. reinforcement has been used.

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CONCRETE
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APRIL 1923 VOL. XVIII. No. 4

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

ALL-CONCRETE ROAD SPECIAL NUMBER.

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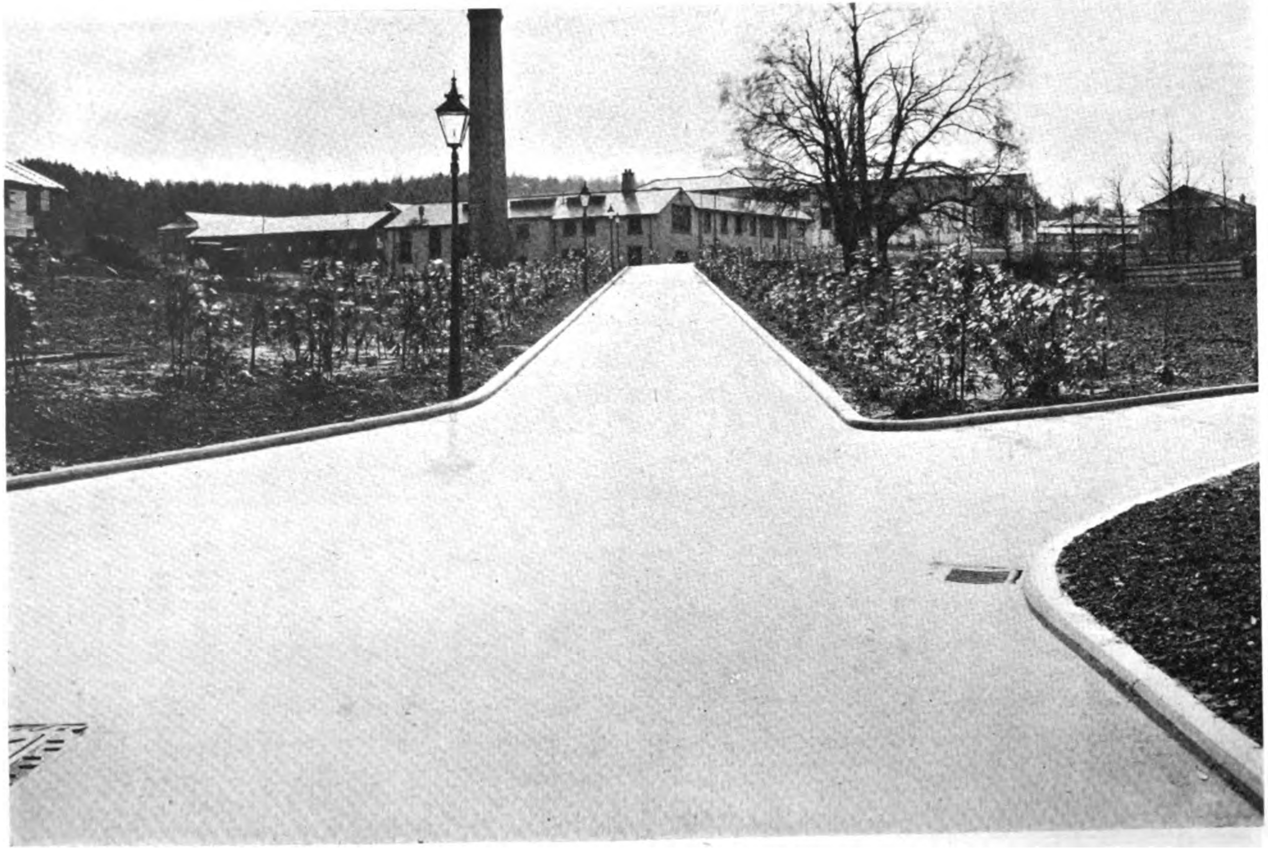

Reinforced Concrete Road at King George V Sanatorium, Godalming.

[This two-course road is 6 in. thick, consisting of a bottom course, 5 in. thick, composed of a 4 : 2 : 1 mixture of 2-in. local stone, 1½-in. granite, pit sand, and Portland cement, and a 1-in. top course composed of a 4 : 2 : 1 mixture of ⅜-in. granite, pit sand, and Portland cement.]

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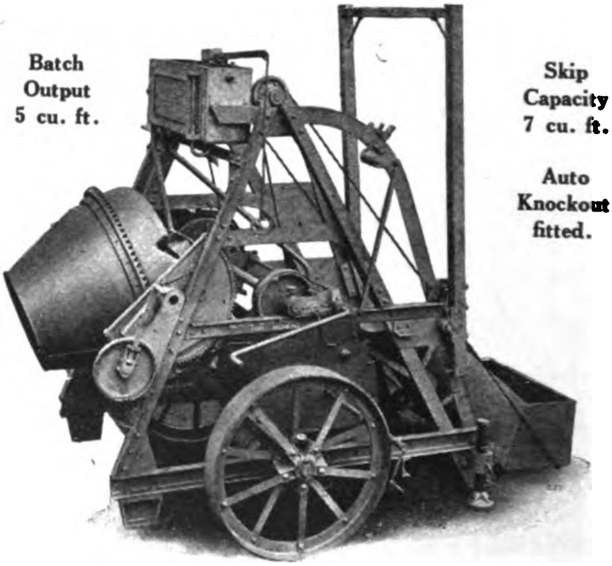

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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume XVIII. No. 4. LONDON, APRIL, 1923.

EDITORIAL NOTES.

THE FUTURE OF ROADS.

OWING chiefly to the development of the motor lorry as a convenient means of transporting heavy loads, a development largely helped by high railway rates forcing manufacturers to look in other directions for the carriage of goods, all classes of roads are now being used to an extent and by a type of vehicle hardly dreamt of when the roads were first built, including the chars-à-bancs, which are playing havoc on rural roads. We are thus faced with a network of roads, the large majority of which are not suited for and were not built for modern traffic, and on the successful solution of the problem of constructing roads to carry heavy and fast vehicles, at an outlay which will not be prohibitive either in first cost or in upkeep, the prosperity of industry, and therefore of the nation, depends to no small degree. The position is a serious one. Enormous sums are now being spent on roads which are in pot-holes a few months after they are laid, and yet the tendency will probably be for vehicles to become heavier and faster in the future, and it would mean incalculable loss if transport were to be limited by the inability of the roads to carry it. The development of industry which took place in the nineteenth century would have been impossible without the contemporaneous growth of the railway systems; and present-day conditions, under which small towns often with inadequate railway facilities are becoming manufacturing centres, together with the advantages of door-to-door delivery without the necessity of re-handling, point to the highways of the country being almost as important in the twentieth century as railways were in the nineteenth.

The importance and urgency of the problem are recognised by the Government, and by local authorities, but is it quite certain that the money of the heavily-burdened tax and rate payers is being spent in the right direction ? The enormous sums spent annually on repair works hardly suggests that this is so. New conditions need new methods, and it may be asked whether the responsible bodies are on the right lines in continuing to reconstruct roads on practically the same lines as in the days before the advent of heavy motor traffic. It cannot be denied that many of the roads are almost ideal for fast and light traffic for the few weeks each year immediately after they are re-surfaced, but the lasting qualities are found to be sadly lacking within a very short space of time when they are subjected to heavy traffic. That this is so is no reflection on Borough Surveyors and others responsible, for, of their type, British roads are among the best in the world. But it seems quite evident that the type of road now generally adopted is unfitted to bear the traffic which uses it, and it is with a view to making better known an alternative method of construction which is receiving increasing support whenever it has been tried that we this month give a special supplement devoted to the all-concrete road. It cannot be over-emphasised, however, that although the idea of leaving the concrete bare and using it as a wearing surface is comparatively new the principle is the same as the concrete road foundations which are now laid in practically all streets which carry heavy traffic in cities and towns. The difference is simply that, instead of laying wood blocks, asphalte, or other material to withstand the wearing and disintegrating effects of traffic, the surface of the concrete itself is made sufficiently dense and hard for the purpose. This admittedly entails more careful supervision and workmanship, but as these are confined to the concrete alone they can be set off against the cost of handling and placing the materials generally used for a top covering. To secure the best results it is also essential to keep a new concrete road free from traffic for at least one month, but that period does not compare unfavourably with the length of time it takes to reconstruct a city street when new concrete foundations are put in, for it is seldom less than five or six weeks from the time a road is opened up to the time when traffic is allowed on it again. Indeed, if the period be shorter there is a danger that the new road will soon be uneven, for whatever the covering the body of the road consists of concrete, which must be given time to set.

The greatest argument in favour of the all-concrete road is its low cost of maintenance. As will be seen from the statements given in this issue by Borough Surveyors and others who have laid this type of road the cost of upkeep over a period of years has been either a very small sum or nothing at all: probably the oldest concrete road in Gt. Britain, a street a quarter of a mile long in Edinburgh, cost only £40 during forty-seven years; the main road from London to Colchester, built with a bare concrete surface in 1915, cost nothing for repairs during the first five years after it was laid; an all-concrete road in Exmouth cost less than £5 for upkeep in five-and-a-half years; an all-concrete road at Padstow required no expenditure whatever for upkeep during six years; a works road at Willesden, carrying 13 ton loads, cost nothing for maintenance during three years; and so on. Many of these roads are subjected to very heavy traffic, and, we think it will be admitted, prove that the all-concrete road is far past the experimental stage; and the remarkable way in which its growth has increased from the dozen or so roads laid before 1918 to the total (including works roads) of over 300 in 1922 shows that it has come to stay. It seems that the all-concrete road is now passing through the stage which took place in America twenty years ago, for since its introduction into that country in 1910 over 25,000 miles have been laid.

The cost of the concrete road necessarily varies according to whether a suitable aggregate is readily obtainable in the neighbourhood; in places where this is the case some strikingly cheap roads have been built, such as the concrete section of the Great West Road, in Middlesex, where the cost of an 8-in. reinforced concrete road worked out at 8*s.* 5½*d.* per yd. super, and the concrete section of the Cambridge Road, Edmonton, built last year, cost 10*s.* 6*d.* per yd. super.

A final point in favour of this type of road is the rapidity with which it can be constructed. With good organisation, a steam-driven mixing-and-placing plant, and a narrow-gauge track on which bogies keep the plant well supplied with materials, the North London arterial roads, 24 ft. wide, are being built at the rate of 180 ft. per day.

HARDENING CONCRETE SURFACES.

As mentioned elsewhere in this issue, the all-concrete road is nothing more than the bed of concrete which is laid under practically all urban roads carrying heavy traffic with the difference that the traffic is borne by the concrete itself and not by a covering of wood blocks or asphalte. The problem, therefore, is simply that of obtaining a surface to the concrete which will resist the wearing and disintegrating effects of traffic. The first essentials are good materials and good workmanship, but the surface of concrete, whether for roads, floors, or other purposes, may be hardened by applying to it a solution of sodium silicate. This material acts by converting the hydrate of lime that is liberated during the setting of cement into silicate of lime, which has the faculty of hardening. The concrete is only hardened to the extent to which the sodium silicate solution penetrates. Sodium silicate is not a proprietary article, and can be obtained in bulk from any manufacturing chemists. There are, however, several forms, and for hardening concrete the most useful form is that known in the trade as the "3·3 ratio," which means there are 3·3 molecules of silica present per molecule of soda. This material is generally sold in the form of a concentrated solution, and for application to concrete one gallon of the solution should be mixed with five gallons of water. A convenient way of applying the solution is through the rose of a watering-can, and no more should be put on the concrete than can be absorbed. Three applications at intervals of twenty-four hours are desirable. Sodium silicate is also known as "water glass," and can be purchased retail under this name.

THE EFFECT OF WATER-CONTENT ON THE STRENGTH OF CONCRETE.

"In certain instances a 1 : 9 mix is as strong as a 1 : 2 mix, depending only on the water content," is one of the striking conclusions arrived at by Professor Duff Abrams as a result of some 50,000 tests undertaken at the Structural Materials Research Laboratory, Chicago, an account of which, together with the conclusions arrived at, are embodied in a Bulletin issued by the Laboratory, and given in part in this issue. The frequency with which we have urged that the smallest quantity of mixing water should be used compatible with a workable concrete is fully justified by these tests, which have led the author to the following conclusions: An excess of one pint over the necessary amount of water reduces the strength of the concrete to the same extent as if 2 lb. or 3 lb. of cement were omitted from a one-bag batch. Provided a first-class cement be used, the water-content is shown to be the most important factor in a concrete mix, inasmuch as small variations in the quantity of water cause a much greater change in the strength of the mix than similar variations in the amount of cement and the size and grading of the aggregate. These tests give a new conception of the function performed by the constituent materials, and show that the use of coarse, well-graded aggregate results in no increase in strength unless advantage is taken of the fact that the amount of water necessary to produce a plastic mix can thus be reduced; also, the use of more cement produces no beneficial effect other than that which results from the possibility of producing a workable mix with less water. The reason a rich mixture gives a higher strength than a lean one is not that more cement is used, but because the concrete can be mixed (and usually is mixed) with a water-ratio which is relatively lower for a rich mix than for a lean mix. If advantage is not taken of the fact that in a rich mixture relatively less water can be used, no benefit will be gained as compared with a leaner mix. The mere use of richer mixes has encouraged a feeling of security, whereas in many instances nothing more has been accomplished than the wasting of a large quantity of cement owing to the use of an excess of mixing water. Workability of concrete is of fundamental significance; this factor is the only limitation which prevents the reduction of cement and water to much lower limits than are now practicable. The present practice of specifying definite gradings for aggregates and carefully proportioning the cement is an absurdity while the proportion of water is left to chance; it would be more correct carefully to measure the water and guess at the amount of cement.

Coming from such an authority as Professor Abrams these conclusions deserve serious study by all interested in cheapening concrete construction. As pointed out in our January issue, Portland cement as manufactured to-day has probably twice the strength of that manufactured twenty or thirty years ago, and there is no doubt that a full realisation of this fact, together with careful proportioning to ensure that the mix contains the right quantity of water to produce the strongest concrete with a minimum amount of cement, would lead to very considerable economies.

THE UNITED STATES STANDARD SPECIFICATION FOR CONCRETE.

THE proposed Standard Specification for Concrete and Reinforced Concrete which has been drawn up by a representative committee in the United States was referred to in our issue for April, 1922. The Specification has been submitted to criticism from various quarters, and the committee now recommends certain modifications. One of the chief stumbling blocks to contractors in the Specification was the clause requiring that samples of concrete taken from batches being put in the work should have a specified crushing strength at twenty-eight days. The results of such tests not being known for twenty-eight days involved that any concrete shown by this test to be faulty would in many cases be covered with much superincumbent material, and very heavy expense would be incurred in removing work condemned, perhaps on tests which, the contractor might argue, were themselves faulty or badly executed. The Specification has now been altered so that satisfaction is given when the average of all concrete tests made on each class of concrete is equal to or better than the requirements for that class; but at least 75 per cent. of the test specimens shall show a strength greater than 80 per cent. of the specified requirement. The recommended slump for hand-finished roads and pavements is reduced from 4 in. to 2½ in. in order that the quality of road concrete should be maintained at the highest possible point. This is interesting as a recognition of the value of keeping the proportion of water in concrete at its minimum. When concrete mixers are used the time of mixing is reduced from 1½ minutes to 1 minute. Slag is inserted in the definition of aggregate as one of the typical aggregates.

AMERICAN METHODS IN CONCRETE WORK.

By ALBERT LAKEMAN, M.S.A.

INTRODUCTION.

CONCRETE work in America is usually executed speedily and with the use of extensive equipment for handling the raw materials and mixed concrete, and thus the methods employed generally should prove interesting. In the chief industrial district of the United States the winter is very severe as compared with that prevailing in this country, and as the period extends over at least four months of the year the time available for executing concrete work under satisfactory conditions is limited; this, together with the fact that the schemes are usually extensive in character, has materially assisted in the development of equipment which facilitates execution. Generally speaking, the quality of the work carried out is good, and the large volume of concrete which is frequently placed in a short time is not the result of rushing forward regardless of proper care in execution but rather the outcome of well-arranged methods which include sufficient equipment.

POURED CONCRETE.

Extensive use is made of towers and chutes for pouring concrete direct on to the work, and considerable study is given to the lay-out and arrangement of the storage of the raw materials close to the mixer to enable economical and speedy handling. On a large scheme covering a wide area of operations it is usual to provide two or three main concrete-mixing stations to supply the whole of the work, that in the immediate vicinity being poured direct and the remainder being concreted from trains which run past the mixing stations for loading up with the mixed material.

Figs. 1 and 2 show a typical installation in operation the units of which consist of storage bins, cement shed, water tank, mixer, stationary derrick, tower and chute. The mixer, which is of the 1-yd. capacity batch type, is placed at ground-level with a raised platform at the feed hopper level and immediately above are the storage bins for the aggregate. The aggregate is fed into the mixer-hopper

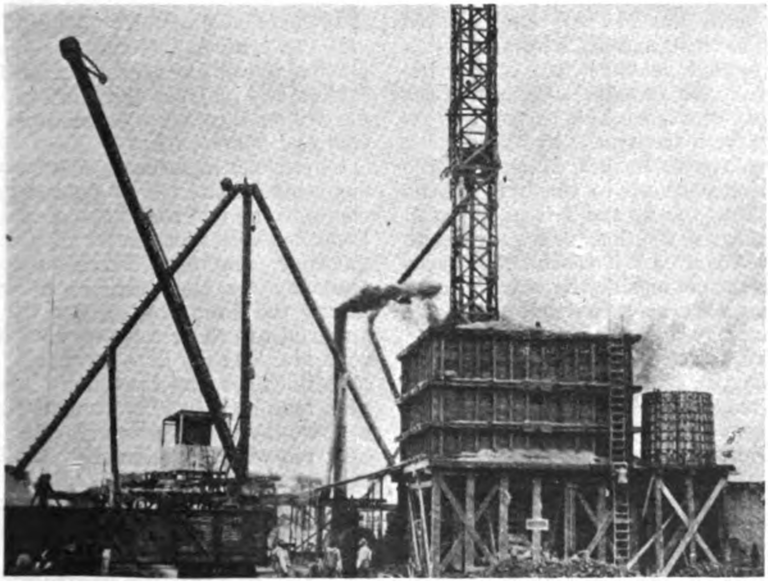


FIG. 1.—CONCRETE MIXING PLANT.

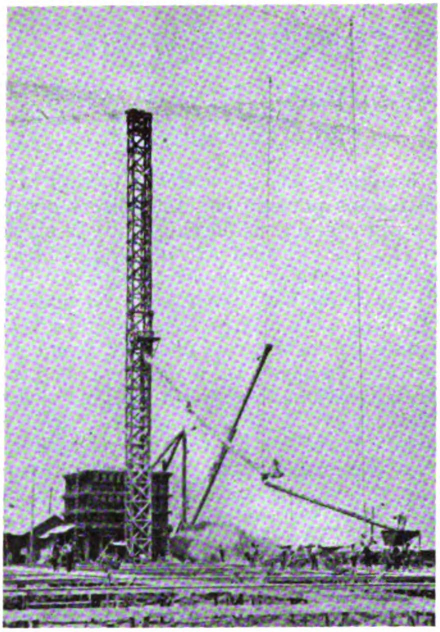


FIG. 2.—CONCRETING TOWER AND CHUTE.

through a hopper-gate in the bottom of the storage-bin, and no handling is necessary by the man in charge of the gauging. These storage-bins are of large capacity and constructed with balk timbers with flat bottom and vertical sides. The installation is served by a standard-gauge track which allows the materials to be brought right up to the point of storage. The aggregate is unloaded from the trucks by the derrick operating a grab bucket which lifts the material straight up to the bin; if the latter is full, the material is piled up in the vicinity of the derrick for raising to the bin as required. The cement storage-shed is placed alongside the railway track and the cement can be unloaded from the trucks directly into the shed. This shed is also adjacent to the platform at the mixing-hopper level, and a sloping runway connects the two to allow the cement to be taken from the store direct to the mixer, either by hand labour or with a conveyor belt which extends down the centre of the store. The large tank for the storage of water adjacent to the aggregate bin is also shown in *Fig.* 1, and this is provided to ensure a reserve supply and also a constant head of water which will fill the tank on the mixer within a specified time. It frequently happens on large construction work that the water supply at certain points is variable and unreliable owing to small-diameter pipes being employed, and many draw-offs are installed which may be constantly opened and shut; it is, however, a poor policy which allows the output of mixed concrete to be affected by insufficient water supply through such causes. The hoisting tower, 160 ft. high, is shown in *Fig.* 2, which also gives a view of the concrete chute and the method of carrying this. The discharge from the mixer is made into a steel hoisting-bucket having a capacity of 1 yd. cu., and this is hoisted up the tower to the required level, where it is automatically tipped to discharge the contents into the hopper provided at the top of the chute, from whence the concrete flows by gravity to the required point. The power for the mixer, derrick and hoist is usually supplied by one steam boiler placed in any convenient position near by. The capacity of an installation of this type is 1 yd. cu. of mixed concrete per minute, or 60 yds. cu. per hour. It is seldom, however, that this output can be maintained throughout the whole day, but it is quite usual for the output to reach 430 yds. cu. in an eight-hour day. In order to achieve this result it is essential that the organisation throughout be good, and the arrangements for receiving and placing the mixed concrete must be considered and planned to avoid breakdowns or delays through a weak link in the chain of operations.

Objections have been raised by many engineers to the use of poured concrete owing to the fact that the material has to be mixed with more water than the ideal quantity, and also because there is a tendency for the particles to separate during the flow, thus causing the coarse material to collect together and giving a concrete lacking in uniformity. These objections are, however, more imaginary than real, as it has been found that with a reasonable inclination to the chutes concrete of the consistency suitable for reinforced work, which is always rather more "sloppy" than that suitable for mass work, will flow without trouble, and the separation of the particles is corrected by the installation of a receiving-hopper, as indicated in *Fig.* 3. The materials

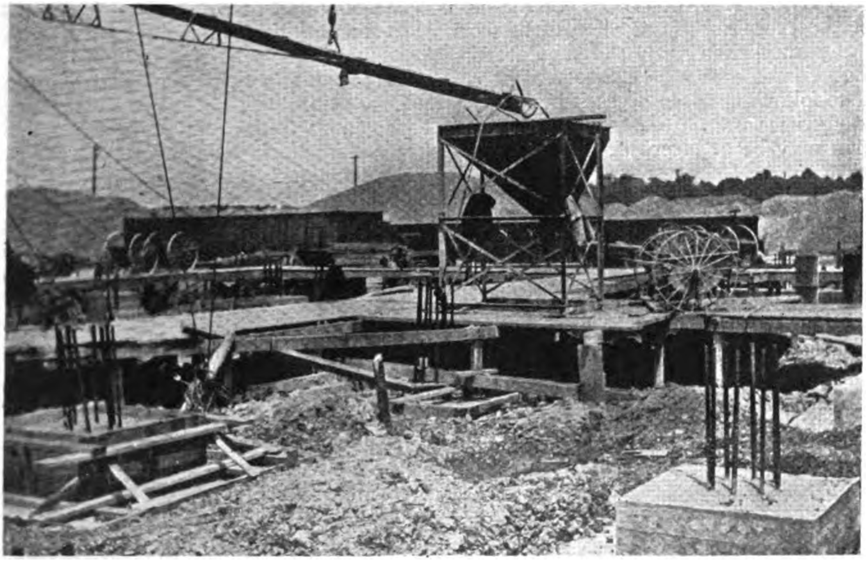


Fig. 3.—Receiving Hopper for Poured Concrete.

come together in the hopper and are then loaded into barrows or buggies for transport to the concreting point. When a large pier foundation or machine-base is being concreted the material can be spouted directly into position, as the excavation will act in the same manner as the receiving hopper, and the tamping and spading generally will mix together any particles which tend to separate.

The methods adopted in connection with poured concrete in America can possibly be best illustrated by the description of the construction of a large reinforced concrete building which was speedily erected, because the notes will then cover an actual accomplishment and cannot be considered as simply dealing with the theory of pouring concrete under ideal conditions.

CONSTRUCTION OF A LARGE WAREHOUSE.

The building of a large warehouse in reinforced concrete was part of an extensive scheme in connection with the erecttion of a new factory in the State of New York, and while excellent speed was accomplished it must be borne in mind that other work on a large scale was proceeding on other parts of the site; thus the supervision and organisation was not confined to this one structure alone. The building is 546 ft. long, 162 ft. wide, and four stories high, and provides a total of approximately 353,800 sq. ft. of floor surface, or about 8 acres for the whole building. The entire construction was of reinforced concrete and in its erection over 14,580 yds. cu. of concrete were necessary, while the complete structure was put up in fifty-five working days.

The speed accomplished was due to the efficient lay-out for the mixing and handling of the raw materials and mixed concrete, the organisation of the workers, and the adoption of well-designed form work. The ground for the foundations was broken on June 3, 1920; the first concrete was poured on June 5; on July 30 the roof slab was completed. The parapet and pent-house walls were finished on August 4. There were four Sundays on which no work was done, and four rainy days on which so little work was executed that it could not be noted on the official records. Deducting these days but fifty-five days are left on which work was done from breaking the ground to pouring the last concrete for the roof parapet and pent-house walls. This period includes some partly rainy days on each of which several hours were lost. The date of completion was not fixed beforehand, and no attempt was made to break any record; therefore practically all the work was accomplished with the regular day-gang.

The following table gives some data as to quantities and dates of placing concrete which may be of interest:—

Item.	Concrete. Cubic Yards.	Commenced Pouring.	Completed Pouring.
Foundations	2,947	June 5	June 21
1st floor and ground floor columns	2,977	June 28	July 10
2nd floor and 1st floor columns	2,850	July 9	July 17
3rd floor and 2nd floor columns	2,759	July 16	July 24
Roof slab and 3rd floor columns	2,778	July 23	July 30
Roof parapets and pent houses	275	July 31	Aug. 4

The floors were of the standard American two-way flat-slab system supported on reinforced concrete spiral-hooped columns having circular caps with drop-heads. The ground floor is 6 in. thick reinforced with expanded metal, and the upper floors and roofs are 7 in. thick reinforced with ordinary round rods. The columns are spaced 18 ft. by 18 ft. 10 in. on centres, giving 224 interior columns on each floor or a total of 896 interior columns in the entire building. Two reinforced concrete staircases and two large freight elevators are provided, in addition to which special conveyors for the transport of the goods inside the building are arranged.

The columns on the ground floor are 26 in. diameter and 14 ft. 4 in. high; those on the first floor are 24 in. diameter and 12 ft. high; and those on the second and third floors are 20 in. in diameter and 12 ft. high. Eight lines of vertical reinforcement were provided, these being four 1-in. sq. bars, four ¾-in. diameter round rods on the ground and first floors; eight ¾-in. diameter rods on the second floor; and four ¾-in. and four ½-in. diameter rods on the third floor. Spiral hooping formed with ⅜-in. diameter steel was provided on the three lower floors with a pitch varying from 2 in. to 2½ in., and the top floor was hooped with ⅜-in. diameter ties at 12-in. centres.

The external and corner columns, which were rectangular on plan, were reinforced in a similar manner, but ties at 12-in. pitch were used throughout the

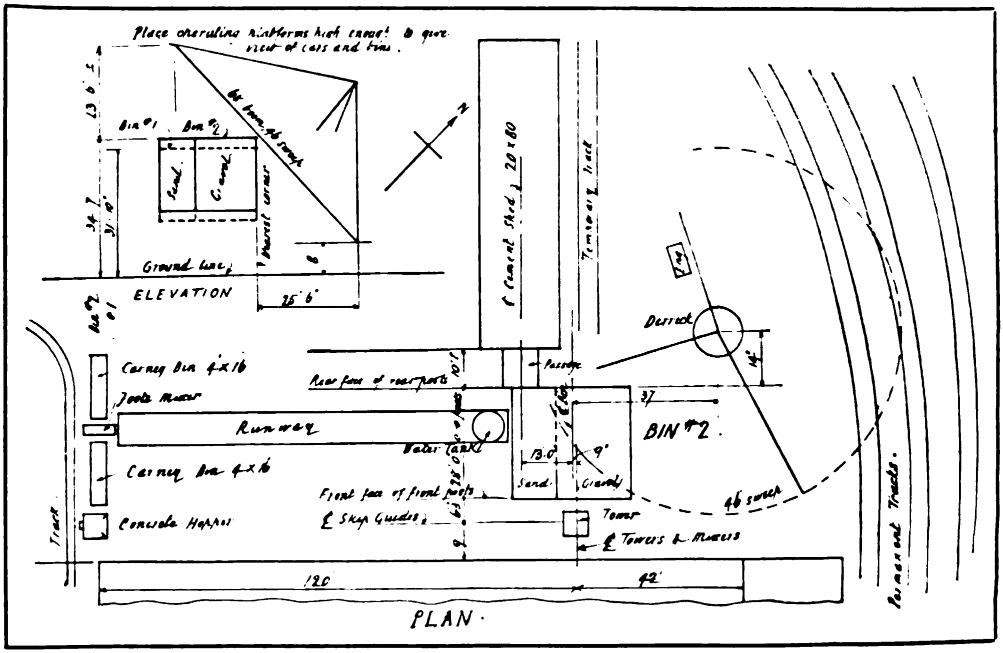


FIG. 4.—LAY-OUT OF PLANT AT NORTH END OF BUILDING.

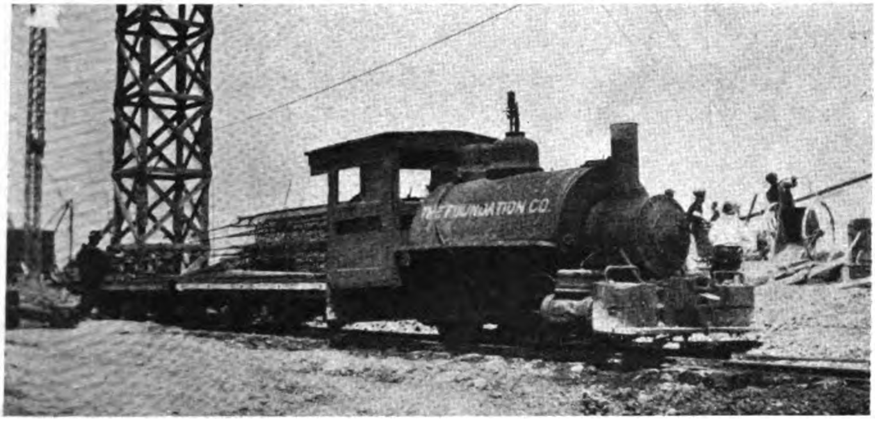


FIG. 5.—REINFORCEMENT DELIVERED ON NARROW-GAUGE TRACK.

height. As the building was 560 ft. long it was deemed necessary to provide an expansion joint across the full width of the building, and this was accomplished by constructing twin columns and piers at about the centre of the structure with a 2-in. clear space between, and twin beams were formed at this point so that the floor slabs in the two sections were separated and entirely independent of one another. The 2-in. joint, which, of course, extended through the external walls, was masked by a concrete pilaster attached to one section only, and by its overlap on the other section the joint was not visible and did not mar the general appearance of the structure.

The concreting plant was arranged to give two systems, one at each end of the building, and a lay-out of the towers, bins, and sheds is indicated in *Fig.* 4, which is a plan of the installation at the north end of the structure. Each plant consisted of a 1-cu. yd. concrete mixer placed at the base of the hoisting tower, which was 150 ft. high, and immediately under the gravel-bin. The gravel-bin, which was of large capacity, was filled by means of the derrick shown adjacent to it, this derrick having a 1-cu. yd. clam-shell grab-bucket and a wide sweep which enabled it to grab up the aggregate from the trucks on the railway tracks adjoining and deposit it direct into the bin. The aggregate passed from the bin into the measuring hopper over the mixer by gravity, and the quantity was controlled by a hopper-gate easy of operation. As will be seen from the lay-out plan, the cement-shed was also placed close to the concrete mixer with a runway connecting the two. The railway tracks were arranged to feed both aggregate and cement right up to the storage point, and no re-loading for transport was necessary. These concreting plants were not only employed for use during the construction of the warehouse, but were used throughout the scheme for general concrete work by spouting from the hoisting towers to concrete-hoppers placed at the corners of the building, from which the mixed material was loaded into special concrete-cars operated over a temporary industrial railway running to all parts of the site.

In addition to the two towers erected for hoisting concrete, provision was made for hoisting the reinforcement, forms, and miscellaneous material by building three other towers, each 100 ft. high, spaced along one side of the structure and adjacent to the industrial track. These towers were equipped with swinging arms near the top which allowed the material to be hoisted with a steam-driven winch, and the arm was then swung back over the required position and the material lowered down on to the floor. *Fig.* 5 shows how reinforcement was delivered on the industrial track to the foot of one of the material-hoisting towers, from whence it was lifted directly to the work.

The forms used for the concrete work were very simple, those for the floors being built up with ⅞-in. yellow pine dressed boards made up into sections and supported on plain bearers and struts,

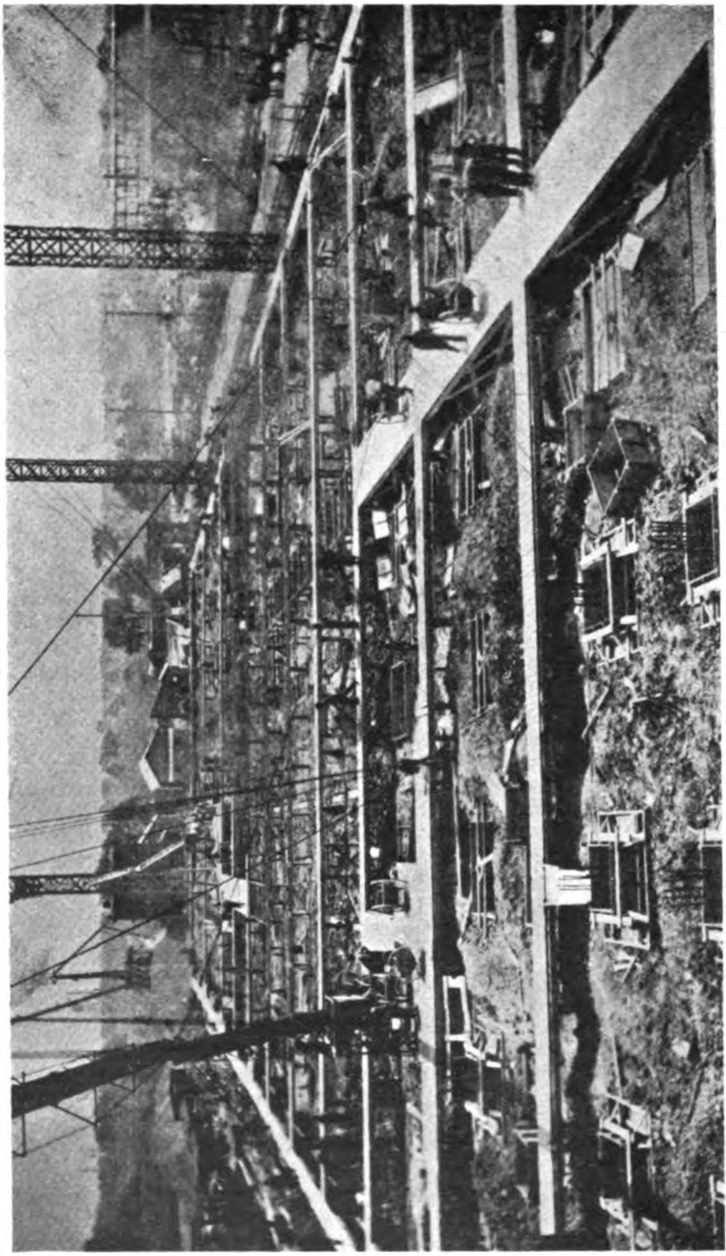


FIG. 7.—GENERAL VIEW, SHOWING RUNWAYS.

while the forms for the exterior columns and walls were built up with 2-in. timber. These wooden forms were all used, on an average, twice each, and many were afterwards adapted and used again for concrete work elsewhere.

The forms for the interior columns were made of collapsible thin sheet-steel and were hired from the makers. A view of one of these forms in position is given in *Fig.* 6. They were used approximately three times each. The method of handling and placing the concrete is of interest in view of the speed which was attained, and a detailed description is therefore given. The concrete was mixed at the ground-level, hoisted to the required height in a 1-cu. yd. capacity steel bucket operating up and down the 150-ft. hoisting tower, tipped into a hopper on the tower, and then discharged into the trussed steel chute which carried the material to another steel hopper at the level of the floor to be concreted. The type of hopper employed is that shown in *Fig.* 3. From this hopper the concrete was taken to the desired point in two-wheel buggies on wooden runways, the latter being one of the features of the scheme making for speed and economy in labour cost, and *Fig.* 7 shows how these were arranged, this photograph being a general view of the warehouse on June 11, six days after the first concrete was placed. In the left foreground can be seen the hopper which received the concrete from the chute and from this hopper—around which a platform was built—the one-way traffic portable transverse runways led to the longitudinal two-way traffic runway. The buggies moved in one continuous stream from the hopper to the place where the concrete was to be deposited and then onward to the nearest transverse runway, and returned to the hopper via the two-way traffic runway—a circular trip which obviated any waiting on the part of the labourers. These runways were very light, being constructed with ⅞-in. boards on 2-in. by 4-in. battens, laid on a skeleton frame with stilts to give a height of about 18 ins. above the top of the foundations or floors. Owing to their lightness and the fact that they were portable these runways were taken from floor to floor, and their initial cost was saved several times over during the execution of the work. A closer view of these runways is given in *Fig.* 8, where the value of the stilting above the floor can be seen and also the continuous route that was provided for the traffic. The concreting was commenced at both ends of the building and worked continuously towards the centre by two distinct gangs of labourers, the carpenters' work being always kept ahead of each gang by a large force of men. The maximum amount of concrete poured in one day on this building from both mixers was over 800 cu. yds., on July 20. The system of working two gangs resulted in a very keen rivalry, which was naturally encour-

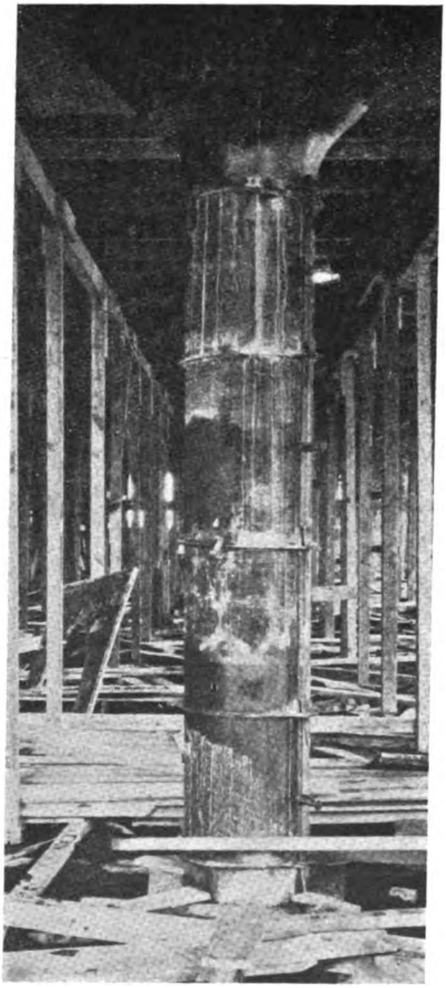


FIG. 6.—STEEL COLUMN FORM.

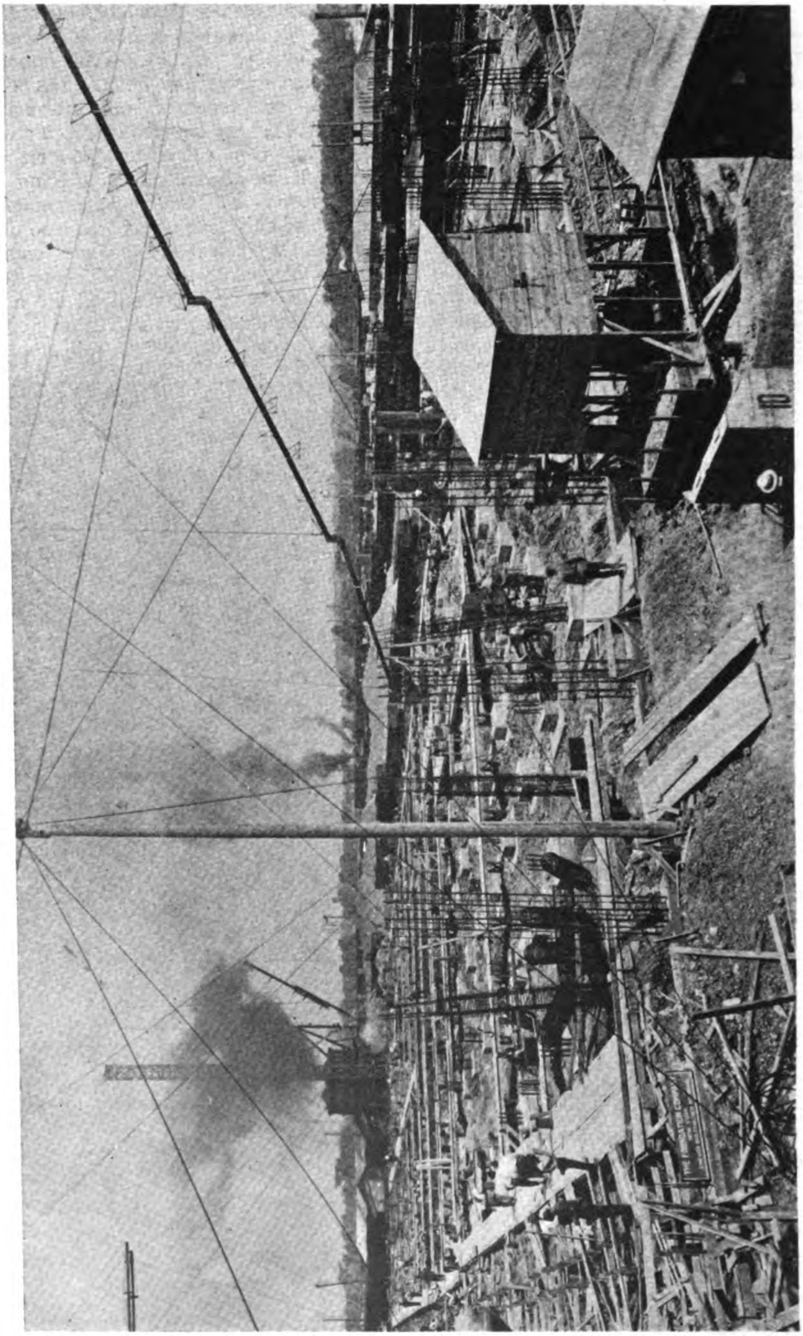


FIG. 9.—GENERAL VIEW OF WAREHOUSE ON JUNE 14.

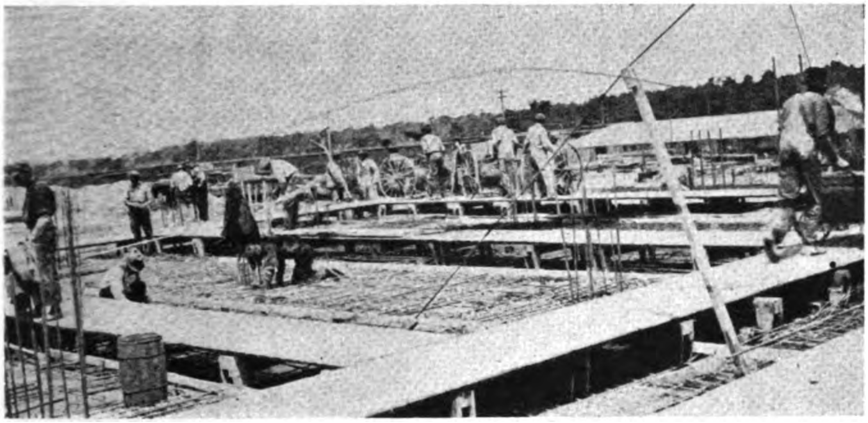


Fig. 8.—Stilts to Runways.

aged by the foremen, and in consequence daily betting was made by the black labour in one gang that they would place more concrete during the working hours than those in the other gang; and it does not require much thought to realise how the speed of the work was thus greatly increased. Black labourers are easily worked up into a state of excitement, and their almost childish delight in racing with the buggies loaded with concrete was a cheerful sight in those days of more or less restricted output. No delay was tolerated by the man operating the hopper at which the buggies were loaded, as the largest bet was often placed by him and he was very keen on giving each man the maximum concrete his buggy would hold in the quickest possible time. The gang at the southern end of the building held the record output, which was 432 cu. yds. from the one mixer in one day of eight hours. A general view of the work (taken on June 14) is given in *Fig.* 9, and another view (taken on July 20) is given in *Fig.* 10.

The method of laying and protecting the floors is of interest as being different from that usually adopted; the entire thickness being laid in one operation. After the forms were in place the pipe-outlet boxes and sleeve-pieces were placed in position together with the cast-iron inserts, and these were nailed to the framework. A typical sleeve-piece is shown in *Fig.* 11, which also illustrates the junction between the steel column-form and the wooden floor-form. The inserts were provided for the attachment of hangers to support sprinkler and heating pipes and also for the conveyors; they numbered approximately 10,000, each one of which had to be placed and secured in its correct position. After the slab reinforcement was placed and wired at the crossing of the rods, the electric light conduits were laid down on top of the steel and properly secured with the outlet boxes extending down to the forms, and the work was then ready to receive concrete.

The concrete was deposited from the buggies to the required thickness and screeded off. The floor finishers then followed with wooden floats, the operation of getting a good surface consisting of dusting neat cement on to the wet concrete to take up the surplus moisture and produce a rich mixture for trowelling. After treatment with wooden floats the floor was left for a period, and before the final set occurred the finishers again went over the whole surface with steel floats to produce a hard and smooth finish. This method eliminates entirely the expense of laying granolithic or other surfacing material, and in addition it has the advantages of giving a surface which is part of the construction and therefore not liable to break up; also, time is saved by completing the whole floor in one operation. When the surface was finished and the final set had taken place, sawdust to the thickness of 1 in. was spread over the floor to protect the finish. The sawdust was kept moist and consequently protected the fresh concrete from too rapid drying under the sun's

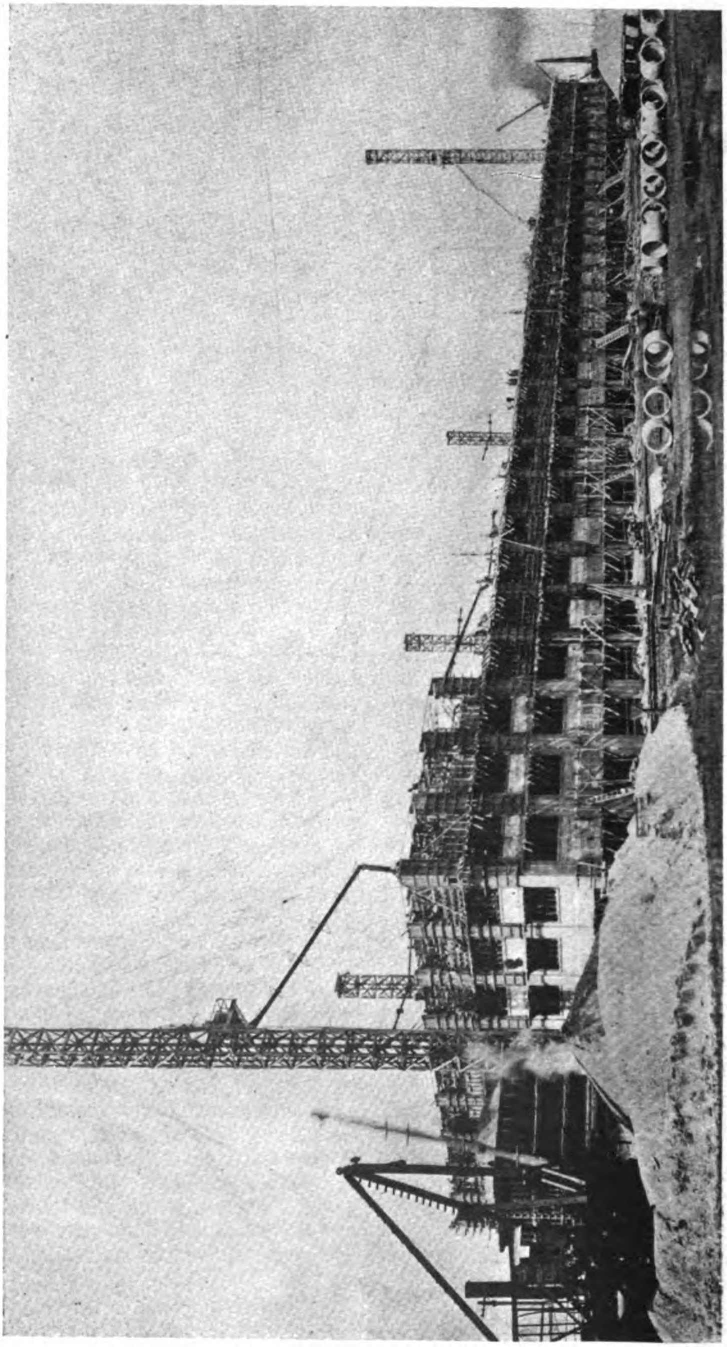


Fig. 20.—General View of Warehouse on July 20.

rays, and in addition prevented damage from tools or concrete which might be dropped from above when executing the next floor.

Fig. 12 illustrates several stages of the work proceeding at the same time. The reinforcement in position ready for concrete is seen in the left centre portion ; to the right of this concrete is being deposited from the buggies ; still farther to the right are two men screeding the concrete ; and at the extreme right, slightly in the rear, are two men floating and finishing the surface. The forms and concrete for the next floor are shown proceeding in the background, while in the foreground the sawdust covering the fresh concrete is seen, as are also the sills for the forms of the next floor. These sills were placed on the sawdust when the floor was 24 hours old, and upon them were placed the 4-in. by 4-in. yellow pine struts to carry the next tier of forms, three rows of struts being provided per span in each direction. An interior view of part of one floor after the removal of the forms is given in *Fig.* 13.

Generally speaking, the time allowed for the removal of the forms was as follows :

Columns—24 hours after pouring (minimum).
Drop caps of columns—48 hours after pouring (minimum).
Beam sides—48 hours after pouring (minimum).
Beam bottoms—14 days after pouring (minimum).
Floor Slabs—14 days after pouring (minimum).

Upon the removal of the forms the concrete finish was generally so good that no surface treatment was necessary internally, while the exterior of the building was rubbed down to improve the appearance.

The temperature ranged during the work on the building from a minimum of 50 deg. Fahr. at night to a maximum of 90 deg. during the day.

Two general views of the structure nearing completion are given in *Figs.* 14 and 15, the former being taken on July 28 and the latter on August 11. In these two views the material hoisting towers can be seen, together with various other features of interest.

There has been a great deal of discussion as to the relative speeds of erection of steel framed and reinforced concrete buildings, but this actual example of the possibilities of rapid concrete construction should go far to satisfy those sceptics who maintain—often without any experience—that steel framing will always be the better method to employ when speed is desired. The writer would be very interested to find an actual example of steel fabrication and erection which excels this reinforced concrete warehouse for speed, bearing in mind that the work was carried out during ordinary working hours and was not simply a record-breaking effort. As mentioned previously, poured concrete is extensively used in America, and on nearly all large schemes the hoisting tower and chute is considered an essential part of the plant to be installed. In the erection of apartment houses of several stories, which are a feature of American cities, a tower in a central position is usually put up, and

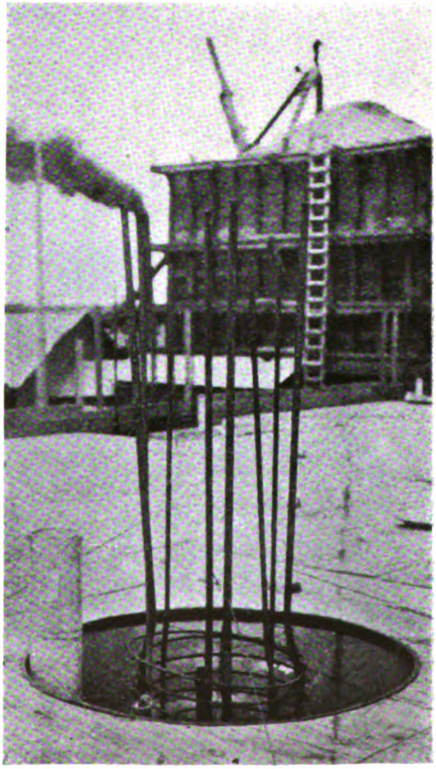


FIG. 11.—SLEEVE-PIECE FOR PIPE.

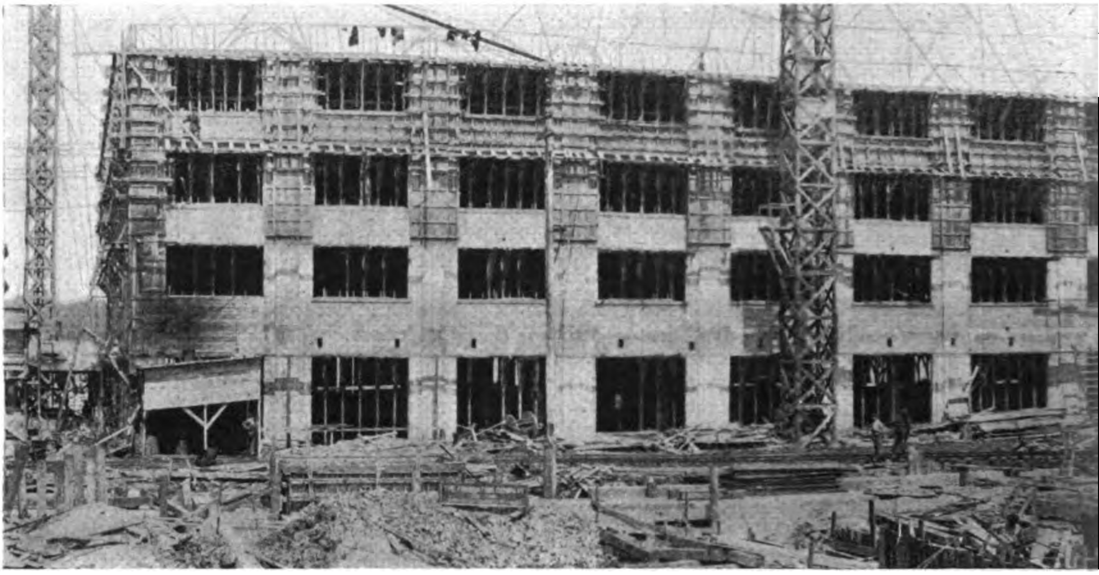


Fig. 14.—Warehous

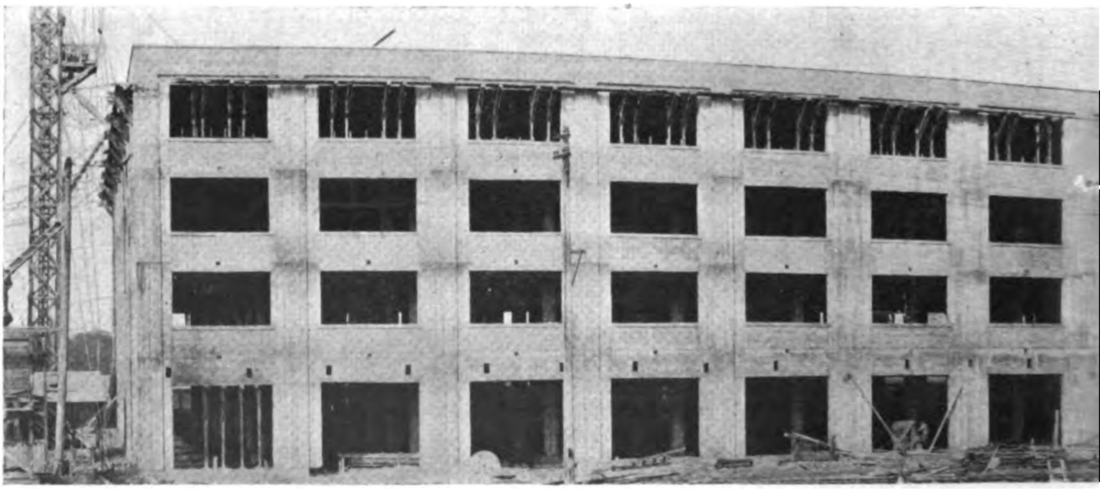
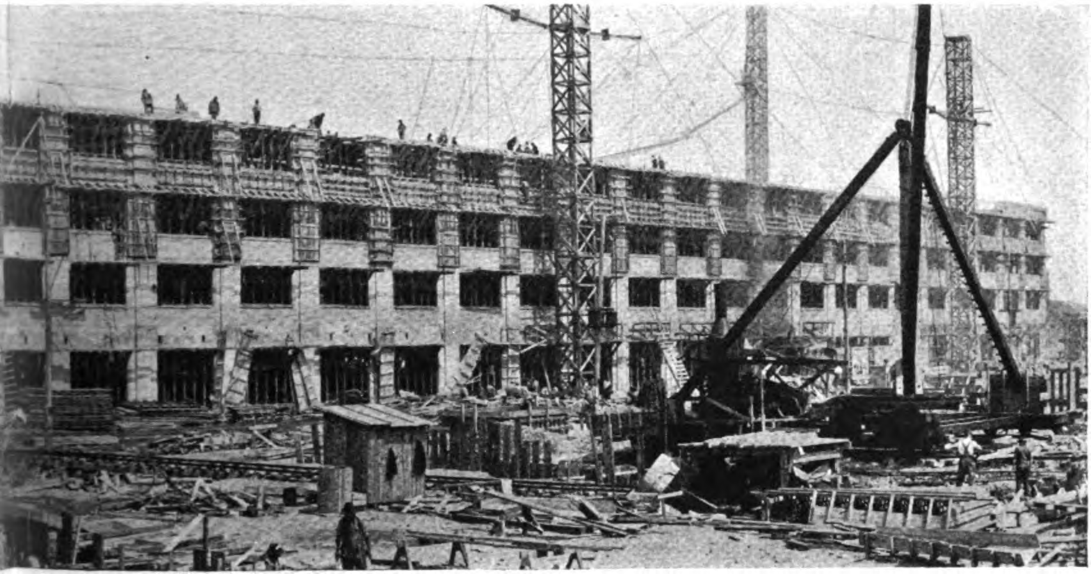
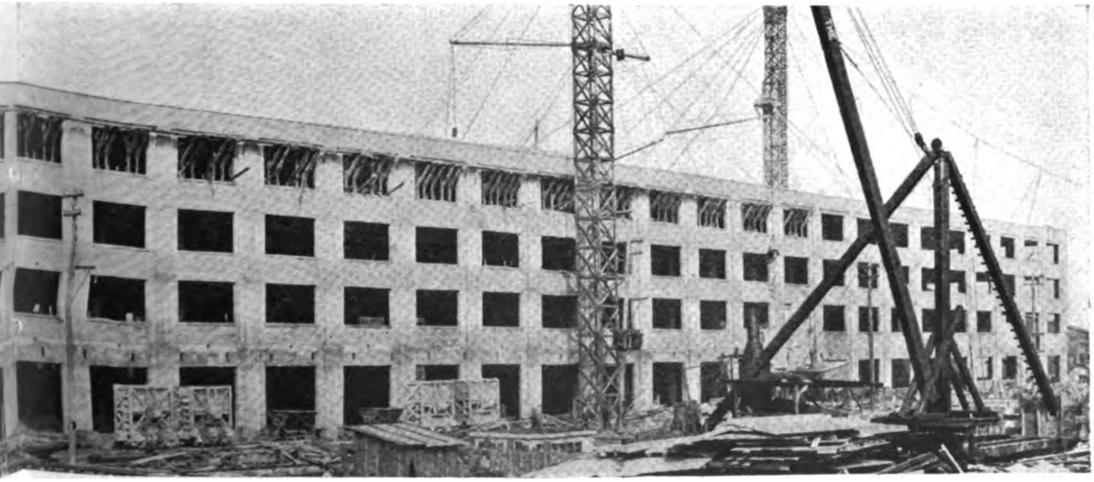


Fig. 15.—General V

WAREHOUSE, NEW YORK STATE, 546 FT. LONG BY



NEARING COMPLETION.



IEW OF WAREHOUSE.

62 FT. WIDE, BUILT IN FIFTY-FIVE WORKING DAYS.

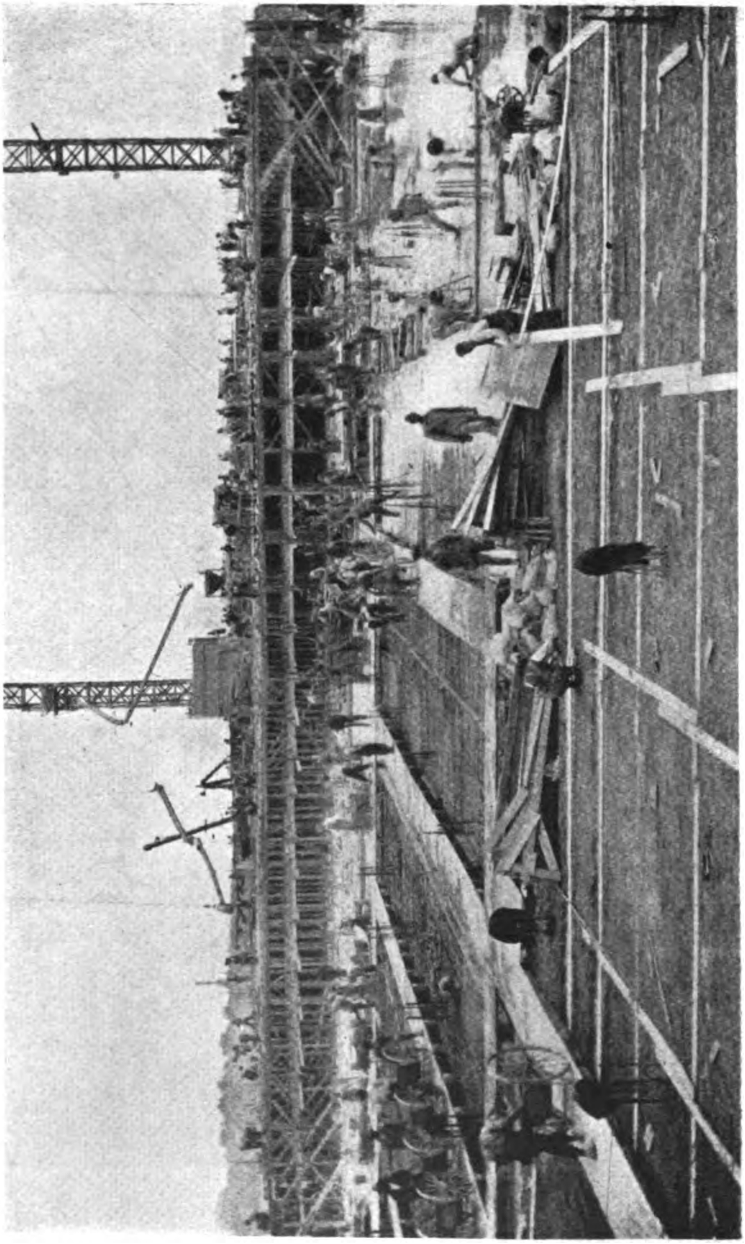


Fig. 12.—General View of Operations.

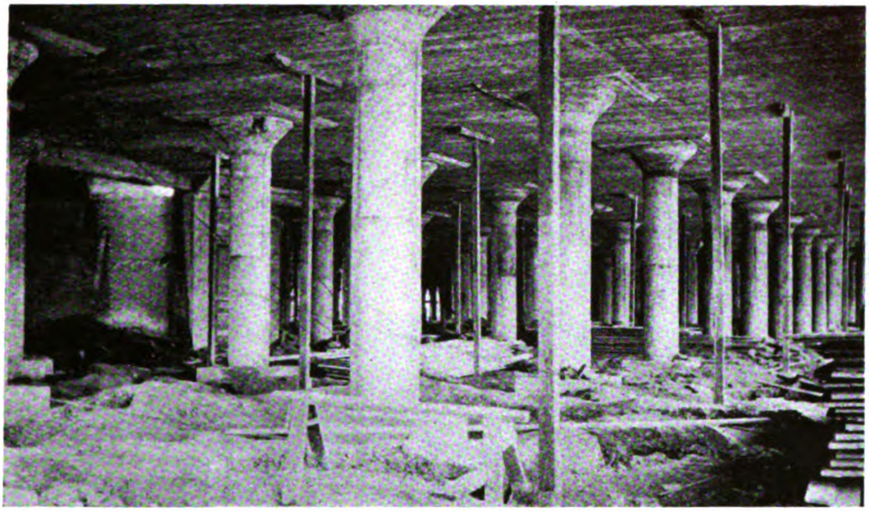


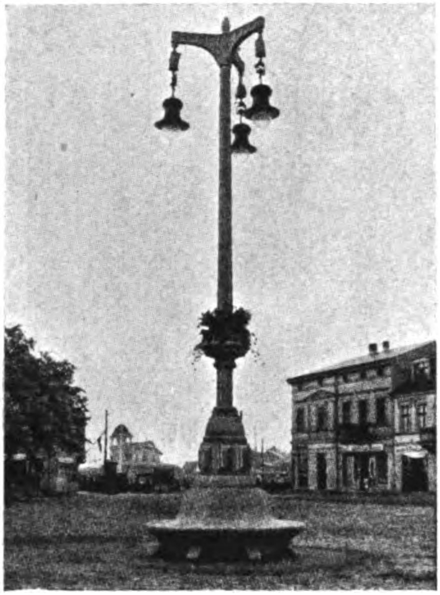
FIG. 13.—INTERIOR VIEW AFTER REMOVAL OF FORMS.

from this one point the whole of the concrete work is controlled. In this country the single-story building will usually be cheaper per sq. ft. of floor surface provided than the multi-story, whereas in America the case is reversed. This may appear curious at first sight, but the fact is a striking proof of the more extensive use of labour-saving plant in American work than in this country. When the work is centralised or concentrated within easy reach of hoisting towers, material hoists, chutes, and other equipment, it will mean saving in time and cost if these are installed, and in American methods considerable attention has been devoted to this type of work, whereas the single-story building covering a large area involves travelling equipment that will not always be justified on account of the light nature of the work to be executed and thus the developments have not been so rapid or successful. Full advantage is not generally taken of hoisting towers, chutes, and similar equipment in this country, and the cost of multi-story buildings involving work of a heavy character is therefore increased over that prevailing for single-story work. This difference, while very marked in the case of concreting towers, also applies to the erection of steelwork, brickwork, and almost all types of construction.

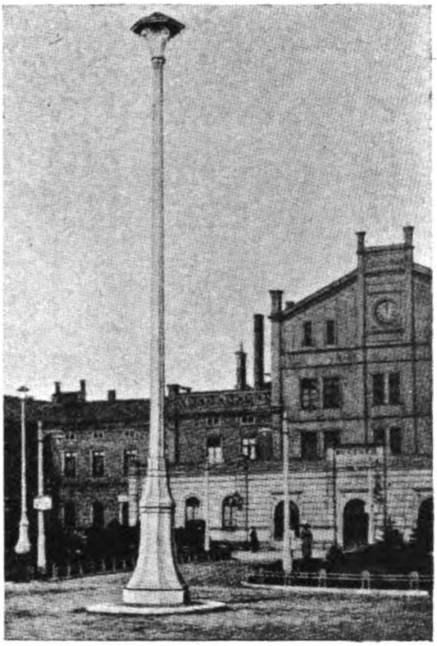
(*To be continued.*)

Long Concrete Arch Bridge.

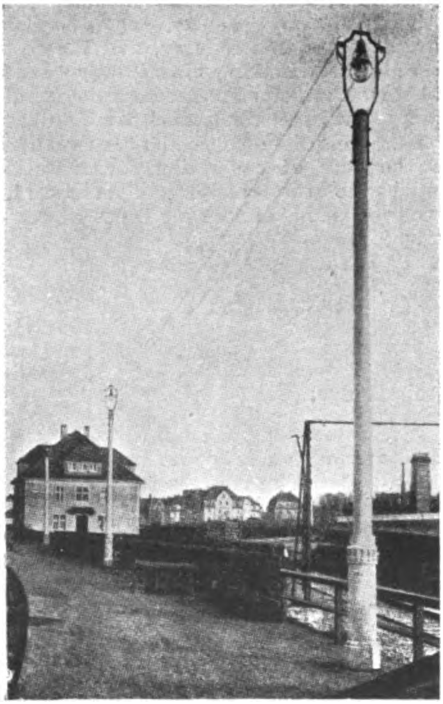
THE new bridge across the Mississippi River at Minneapolis is claimed to be the longest reinforced concrete arch in the world. The main span is 400 ft. long by 88 ft. high, and is flanked by two other arches of 199 ft. The main ribs are 16 ft. through at the springing and 8 ft. at the crown. They are spaced 25 ft. apart in the clear, and thus could be constructed one on either side of the existing steel bridge while the latter was still in service. The old bridge was removed when the floor of the concrete structure was put in place.



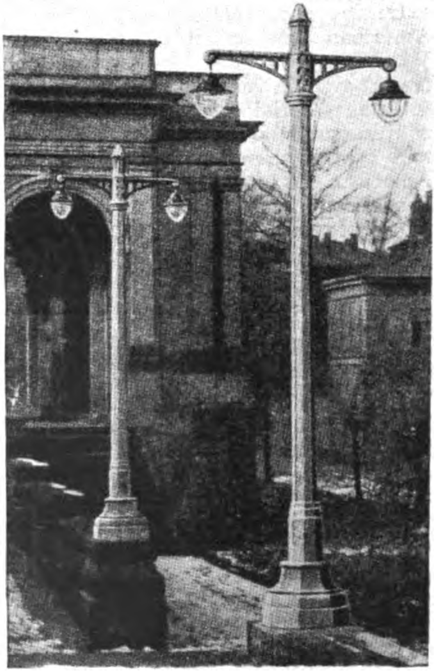
Ornamental Lamp Standard in Swinemünde.



Lamp Standard at Littau.



Lamp-post carrying Overhead Wires.



Lamp Standard in Dresden Cemetery.

REINFORCED CONCRETE MASTS ON THE CONTINENT (*See* p. 225).

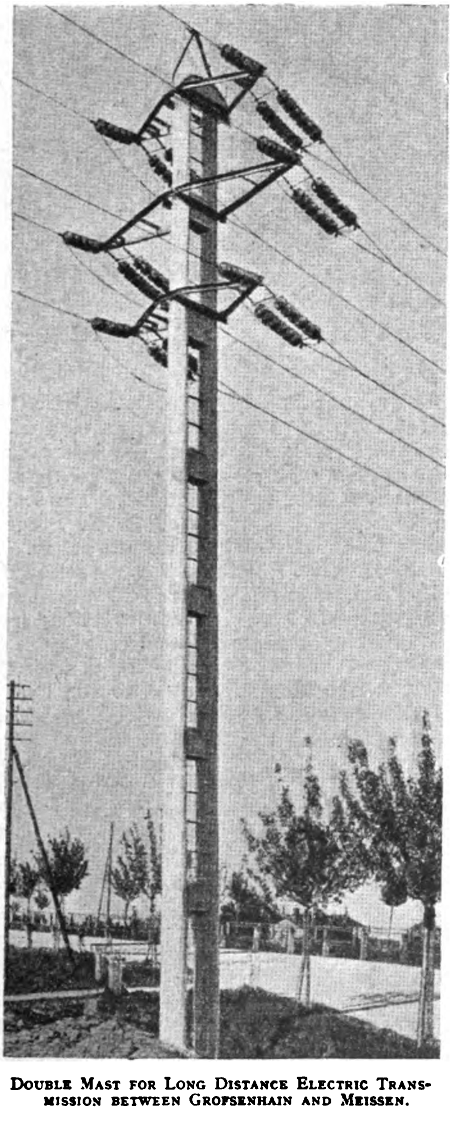
REINFORCED CONCRETE POLES ON THE CONTINENT.

(*Contributed.*)

THE enormous and constantly increasing demand for poles and masts for carrying electric wires and lamps, for signal posts, etc., has led to the adoption of reinforced concrete poles to a very large extent on the Continent, and some of these are shown herewith, together with some notes on the methods of manufacture in Germany.

Reinforced concrete masts made on the centrifugal process are at present manufactured in one piece in any length up to 45 ft. The external diameter at the top of the various sizes of masts varies from approximately 6 in. to 9 in., and at the lower end from approximately 9 in. to 14½ in. The poles are constructed with a wall thickness of from 1$\frac{1}{16}$ in. to 3⅕ in., and are tapered from the foot to the top.

The steel reinforcement is completely embedded in the concrete forming the wall of the mast, and is thus well protected. The reinforcement, as a rule, is formed of longitudinal bars with a tensile strength of from 85,000 lb. to 100,000 per sq. in., combined with spiral linking stirrups both internally and externally. At the crossing points the longitudinal bars and the two wire spirals are connected together by means of iron linking wire.



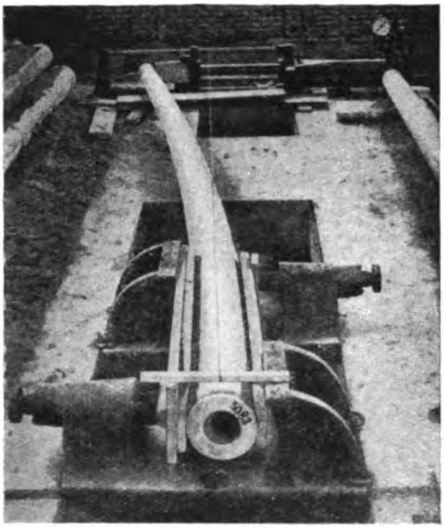
DOUBLE MAST FOR LONG DISTANCE ELECTRIC TRANSMISSION BETWEEN GROFSENHAIN AND MEISSEN.

These concrete pipes, which are manufactured in all sizes, are used for such purposes as conduit pipes; sluice pipes (especially when subjected to great earth pressure or when they have to be laid under difficult conditions); cable and insulating pipes, for electric cables, wiring, etc.; air escapes, drainage pipes, and shaft rings or mines; chimneys; columns for over- or underground workings (which, if necessary, can be filled in); pillars and piles; girders and supports for buildings; supports for mines; telegraph poles; signal-posts; lamp-posts; masts for carrying electric wires, etc.

The following tests have been carried out:—

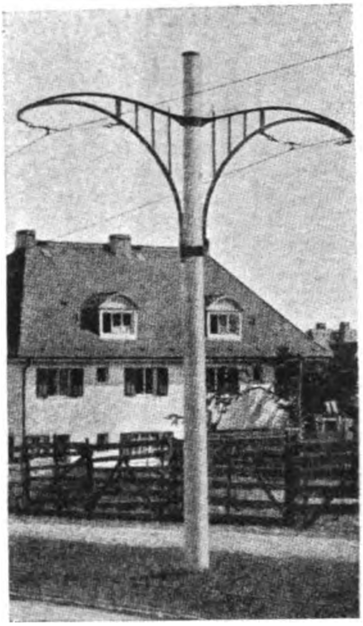
(1) This test was carried out on a pole about 30 ft. long, about 5 ft. being held in a rigid grip from the base. The load was applied at the top of the pole, which was designed for a working load of about 1,210 lb. with a factor of safety of 4.

Load applied. lb.	Deflection. mm.
1,210 . . .	50
4,850 (factor of safety of 4) .	200
7,260 (six times the calculated load) . . .	680



Reinforced Concrete Mast.

[9 metres long, for a calculated load of 550 kilos, with a safety factor of 4, tested with a load of 3,200 kilos (6 times the calculated load). The deflection is 680 mm.]



Tramway Pole, near Dresden.

After the removal of the first two loads, which were applied several times, the pole came back to its normal position; after the third strain, also applied several times in succession, was removed a deflection of 250 mm. remained.

(2) The following tests were carried out by the Royal Mechanical and Technical Testing Institute, Dresden (Saxony):

In Test No. 1 the pole was held horizontally. Without the breaking strain having been reached, a total bend of approximately 3 ft. 8 in. with a single load of 1,210 lb., applied at the top of the pole, was obtained. Upon the removal of the load the bend in a short time disappeared almost entirely.

In Tests Nos. 2 and 3 the poles were held vertically, i.e., in the position in which under working conditions their load would be borne, and tested by the application of a load taken to the top of the pole by a wire rope fixed to the top and led over a pulley and provided at the other end with a receptacle to receive the load. The load was so arranged that it could be progressively increased by 44 lb. or 22 lb. Invariably, when the load was released, an almost perfect return to the perpendicular took place. The deviation from the vertical was almost entirely of an elastic nature. In Test No. 3, after the pole had been subjected to the load during fifty minutes, no appreciable change of shape could be found.

The manufacturers of these poles have made masts 60 ft. long to carry overhead electric wires for a working load of about 60 cwts. with a factor of safety of 3, which equals a breaking load of about 180 cwts.

REINFORCED CONCRETE AND FIRE RESISTANCE.

VI. STRUCTURAL DETAILS.

By J. SINGLETON-GREEN, B.Sc.(Hons. Eng.), A.M.I.Struct.E., M.Am.C.I.

SPALLING.

THERE is a great diversity of opinion as to whether or not a round column is better than a square one. There is expert opinion on both sides, but until more definite information is available the writer feels inclined to give the preference to the round column. The rapid surface expansion of concrete when subjected to a quick fire appears to cause rapid destruction to structural members with sharp corners, and arguing from this it seems as though square columns and beams should be avoided and that only round columns or those closely approaching that shape should be used in the interior of a building.

The safety of beams is scarcely less vital than that of columns, but it is difficult to round them sufficiently to avoid spalling and at the same time properly protect the reinforcing bars. This difficulty can be overcome by using some form of mesh reinforcement surrounding the main reinforcement bars on the bottom and supported by them. The added cost would be slight and the increased security when attacked by fire would be very material.

The writer has shown previously that aggregates which contain quartz are those most liable to cause spalling, and if these aggregates are used then special precautions should be taken.

In June, 1920, a reinforced concrete building in the chemical plant of the Barrett Manufacturing Co., at Frankford, near Philadelphia, Pa., was wrecked by a high-temperature fire from burning naphthalin, a coal-tar product being stored in the building. A general description of the structure is given in *Engineering News Record*, 5–5–1921, Vol. 86, No. 18. The fire started early in the morning and was apparently due to a "boil-over" in one of the tanks. The molten material ran out into the yard of the plant and about 150 ft. distant into the boiler room, where it caught on fire and the fire ran back over the surface of the liberated naphthalin and soon distributed the flames inside the building. The plant watchman was notified of the fire at 5.55 a.m., the naphthalin from the exploded tank being on the ground burning at that time. At 6.30 a.m. the destruction of the building was practically complete; that is, the building was brought to its final failure in about 45 minutes, which is an indication of the quickness and severity of the fire.

Some comments on the design of the building are taken, as follows, from a paper by Mr. W. A. Hull, of the U.S. Bureau of Standards:—

"As has already been stated, this building, first-class in other respects, was by no means of the best type of fire-resistive construction that can be built of reinforced concrete within reasonable cost limits. The gravel aggregate used in this building belongs to the class of aggregates that has a strong tendency to spall when exposed to quick fires. The square columns and rectangular girders made spalling relatively easy. There was, of course, nothing provided in the way of metal mesh or other tie to prevent the loss of the protective concrete by spalling. It has been demonstrated repeatedly by actual fires and by fire tests both in this country and in England that this type of concrete building does not resist quick, hot fires well.

"Whether the building would have endured without serious structural damage if it had been built exactly as it was, except that a more suitable aggregate had been used, is a matter of opinion. At any rate, the fact that it did not endure is not to be regarded as proof that no type of construction could be expected to endure through such a fire, or that it would be advisable to depart from reinforced concrete construction in building to house such an occupancy. The type of construction as well as the aggregate was poorly adapted to resist spalling and other structural damage. The long, slender, square columns and long girders without special ties for the protective concrete made the building particularly liable to such effects.

"In many localities a non-spalling aggregate can be used without excessively increasing the cost of the building. Limestone, either crushed or in the form of limestone gravel, as well as trap-rock and blast-furnace slag, are all so definitely superior to siliceous gravels, crushed sandstone, and crushed granite as to make two distinct classes of aggregates, which for brevity have been called spalling and non-spalling. Where the cost of a non-spalling aggregate is excessive, the severe fire hazard can be met by the use of secondary reinforcement in the outer concrete, together with a reasonable increase in the thickness of the latter. The effectiveness of secondary reinforcement in the form of a light grade of expanded metal placed in the protective concrete has been tried out in fire tests of full-size building columns with promising results. There appears to be no reason for doubting that the fire resistance of girders and slabs made from the spalling types of aggregate could be brought up to any reasonable standard by the proper use of secondary reinforcement with protective concrete of suitable thickness.

" The results of the fires in the warehouse at Galveston show conclusively the great value of flat surfaces without sharp corners of any character, if the greatest protection is to be obtained. The spandrel beams over the door openings in the earlier fire were nearly completely destroyed as far as future effectiveness is concerned, although the flat slab in the immediate vicinity remained nearly intact. Projecting members of every nature, such as drops at column capitals and beams, were destroyed."

CONSTRUCTION JOINTS.

In his report on the fire in the concrete warehouse at Galveston on October 2, 1920, Mr. W. J. Knight [1] says he was particularly impressed by the effect of the fire on the construction joints of all slabs where these joints occur.

He says: " In all cases these joints were perceptible from any point of vantage from one end of the structure to the other where the panels remained in place, each of the joints being emphasised by rough surfaces of the concrete which had spalled off along both sides of it, as well as by the exposure of steel bars. The compartment can be repaired at comparatively little cost, with the exception of the panels through which construction joints extend. These joints, where pouring operation was discontinued until the next day or at a later time during erection, were not specially prepared by roughing the surfaces or by any other apparent means to ensure or contribute to the bonding of new concrete to the old. Examination disclosed very smooth surfaces and the absence of any indication of bond.

" The joints proved to be the weakest part of the entire construction in offering resistance to the effects of intense heat, and have rendered the question of repairing a very difficult one on account of the fact that almost without exception the portions of panels on both sides of the joints are now at different elevations, varying from 1 in. to 1½ in., and in several cases are badly buckled. The almost universal lack of proper care and precaution exercised by engineers and builders in the past in guarding against ineffective and insufficient bond between old and new concrete surfaces seems to indicate that tangible reasons for correction have been lacking. This fire demonstrates beyond doubt the surprising extent of weakness developed by construction joints during exposure to great heat."

FLOOR SLABS.

In the Far Rockaway Fire (1916) the floor slabs proper varied from 4 in. to 6 in. in thickness, and upon these a surface finish of 1½ in. to 2 in. thick was cast after the slabs had hardened. This caused the surfacing to break up when a floor was deflected.

Again, in the fire at the Quaker Oats Company's premises at Peterboro', Ontario, on December 11, 1916,[1] all the floors had been given a 1-in. wearing surface, and practically everywhere throughout the standing portion of the building this 1-in. surface had left the main slab.

WATER-PROOF FLOORS.

The construction of reliable waterproof floors is important. Not infrequently the water damage is far in excess of the fire damage. This is illogical and unnecessary. The value of a waterproof floor (*inter alia*) was demonstrated at an hotel fire in Minneapolis. A hot fire[1] in the ball-room on the 11th floor of the Hotel Radisson, Minneapolis, October 9, 1915, at 3 a.m., destroyed the contents and decorations of the room, with a loss of about 40,000 dollars, yet most of the guests did not know of the fire and the normal operations of the hotel were not hampered. An announcement by the hotel management to its patrons said in part: "Regrettable as this occurrence was, it is, however, not without its gratifying features. The fire developed a very intense heat, sufficient to cause the plaster to crumble and exposed iron to melt. The concrete walls and floor confined the fire to the room and prevented spreading. The fireman's hose played on the fire for upwards of an hour, thereby throwing a large amount of water into the room, yet the floor below suffered no water damage whatever. Because of the fire-proof and smoke-proof elevator shafts and entrance doors, no fire or smoke entered any of the floors below or above the Silhouette floor."

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[1] *Engineering News-Record*, Vol. 85, No. 21, p. 980, Nov. 18, 1920.

[1] See B.F.P.C. Red Book, No. 225.

PARTITIONS.

In his report on the Far Rockaway Fire, Mr. I. H. Woolson[2] states that although the area was not excessive it was quite apparent that had the floor space been divided by partitions of even moderate fire-resisting capacity the fire would have been localised and a large proportion of the damage prevented.

In the Galveston Fire (October 2, 1920) the warehouse was a four-section, two-story building 1,280 ft. long, and from 100 to 300 ft. wide, divided by solid 12-in. reinforced concrete walls into four independent sections each about 300 ft. long. The material stored was sisal, a Central American hemp, and in his report on the fire Mr. W. J. Knight[3] says that buildings designed for housing sisal or other similar materials should have compartments much smaller in size, to lessen the great liability involved by fire. Compartments not longer or wider than 150 ft. would be more conservative.

Mr. W. C. Robinson in his report[4] of the Parker Building fire in New York (1908) states that buildings of large unbroken floor areas filled with combustible contents develop the severest fires and constitute one of the most dangerous sources of general conflagration. Floor areas in buildings of this character should be subdivided by substantial walls having a high degree of fire resistance sufficient to form a positive barrier to the spread of fire. No wood or other combustible material should be employed in the construction of fire-resisting partitions, and all metal supports or reinforcements should be thoroughly insulated from heat. Fire-resisting doors and wired glass in standard metal frames should be used at necessary openings in corridor and room partitions. Provision for expansion in the material used and in metal supports entering into the construction of fire-resisting partitions is essential, particularly where hollow terra-cotta blocks are employed. All fire-resisting partitions should rest on solid incombustible material.

SUPERVISION.

If a high degree of fire-resistance is to be obtained, good supervision is essential. This is seen quite clearly in a fire which occurred in a reinforced concrete warehouse at Far Rockaway, New York, on November 10, 1916. The conclusions are drawn from a report of the fire by Mr. Ira H. Woolson, published by the British Fire Prevention Committee in their Red Book No. 214. In so far as the construction of this building did not follow the design it is an example of the objectionable practice of divided responsibility.

The floor reinforcement was expanded rib metal with 4 in. mesh. The sheets were 32 in. wide, and were laid with a space of 2 in. to 4 in. between each sheet. The design specified that these adjacent sheets of reinforcement should be wired together, but apparently this was not done. The result was that the lack of overlap allowed the concrete to break between each sheet of reinforcement, so that they hung quite disconnectedly. Much of the reinforcement was lying extremely close to the under-surface of the slabs, and did not have proper protection. Bent-up bars, intended to resist negative bending moments, were located on a line with the bottom of the floor slab, instead of being placed near the top surface, where they would have been much more effective and where the design drawings indicated they should be. This same defect existed in all the collapsed beams.

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[1] *Concrete*, U.S.A., December, 1915.

[2] B.F.P.C. Red Book, No. 214.

[3] *Engineering News-Record*, Vol. 85, No. 21, No. 21, p. 980. November 18, 1920.

[4] B.F.P.C. Red Book, No. 132.

OPENINGS.

The folly of erecting a high-grade fire-resisting building in all essential structural features and then allowing its efficiency to be ruined by permitting unprotected openings, even of small size, in walls and floors was strongly emphasised. Even small holes or cracks in exposed walls, floors or partitions are dangerous.[1]

In buildings for mercantile, manufacturing or storage occupancy it is absolutely essential that all vertical openings be thoroughly enclosed in substantial fire-resisting shafts having standard fire doors at all openings or so arranged that the shaft is without openings directly into the various stories. Unprotected vertical openings through buildings are the greatest factor in the loss of life and property by fire, and the proper safeguarding of this hazard demands the most careful attention of all concerned.[2]

Exterior openings in buildings should be thoroughly protected against exposing fires. Universal efficient fire protection of exterior openings will practically eliminate the danger of conflagration in cities.

It is often advisable to protect the window openings with shutters which are arranged to drop by the melting of fusible plugs. The Salem conflagration[3] (June 25, 1914) caused the largest loss ever experienced by the Massachusetts Mutual Insurance Companies of New England. All buildings, with the exception of a reinforced concrete storehouse and an old gasometer, a low, circular structure without windows, which had also been used as a cotton storehouse, were wholly destroyed at the large plant of the Naumkeag Steam Cotton Co. No more complete burning of everything combustible has ever been witnessed. This conflagration, however, produced one very striking example of the fire-resisting qualities of a well-built concrete building with window sashes of solid steel section glazed with heavy wire glass, and window openings protected on the inside with tin-covered shutters hinged to the ceiling and arranged to drop by the melting of fusible plugs. This device worked perfectly in every instance, so that absolutely no fire and practically no smoke was admitted to the building.

As the Naumkeag Company used this building for the storage of very valuable goods, it had installed a full sprinkler outfit, and it is a remarkable fact that although some of the sprinkler heads, set to melt at 166 deg. F., were within 3 ft. of the fusible plugs that held the inside shutters the shutters dropped in time so that not a sprinkler head was started in the entire building. Neither was the white paint, with which the interior walls were covered, blistered, and no damage was done to the 200,000 dollars' worth of finished goods which the storehouse contained. This building, although surrounded on all sides by a sea of flames, came through the fire practically uninjured except for the cracking and melting of the wire glass in a few of the windows and a little smoke damage.

HEIGHT OF BUILDINGS.

The height of fire-proof buildings for mercantile, manufacturing, or storage occupancy should be limited to correspond to the degree of protection the building equipment and the fire department are able to furnish. In other words, if adequate fire protection in any building is not available above a certain height the building should be limited to such height.[1]

High buildings for such purposes should be provided with large, properly enclosed stairways in sufficient number to afford safe exit in time of fire; the buildings should also be provided with outside fire-escape and stand-pipe equipments. They should also be provided with adequate systems of inside stand-pipes equipped with liner hose and nozzles suitable for fire department use, and, in addition, a smaller liner hose and nozzle suitable and safe for use by the occupants. These equipments should be accessible and in sufficient number to cover effectively all portions of the building. They should extend through all stories, and should be supplied from a reliable source of water under adequate pressure in addition to Siamese steamer connections on the outside at street level.

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[1] B.F.P.C. Red Book, No. 214.

[2] B.F.P.C. Red Book, No. 132.

[3] *Concrete-Cement Age*, August, 1914.

[1] B.F.P.C. Red Book, No. 132.

SPRINKLER SYSTEMS.

Fire-proof buildings, no matter how well designed and constructed, do not prevent the destruction by fire of contents in any story; and it is essential that high buildings for mercantile, manufacturing, or storage occupancy be thoroughly protected by a standard equipment of automatic sprinklers.[1] The use of perforated pipe systems should be prohibited, as such systems are unreliable, inefficient, and liable to result in water damages wholly disproportionate to the extent of the fire. Where it is desirable to protect only a part of the building, a system of automatic sprinklers with adequate water supply should be employed, and the portions protected plainly marked at the Siamese steamer connections on the outside of the building.

It was pointed out after the Far Rockaway Fire (1916) that the one fact which stood out above all others was that a suitable sprinkler system would have saved the concrete building and its contents, and probably have controlled the fire in the frame building.

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[1] B.F.P.C. Red Book, No. 132.

BOOK REVIEWS.

Pratique du Calcul du Beton Arme. Part 1. By G. Magnel. 166 pp.

Ghent: Van Rysselberghe & Rombaut.

THIS text-book is written principally for the use of students attending the Ghent University lectures on reinforced concrete, and has for its object to supply a number of practical tables and graphs to obviate unnecessary calculations in every-day problems meeting the designer. It aims mainly at presenting the solutions of these problems in a practical manner for direct use in the drawing office, and that the author on the whole succeeds very well in this praiseworthy attempt is perhaps due to the fact that he spent some years in a contractor's office in London.

In some respects the book is a departure from the traditional reinforced concrete text-book; it is thus devoid of any lessons in statics, which often take up a good deal of space in many text-books. Professor Magnel's students are evidently supposed to know their statics or to obtain their knowledge of them elsewhere, for he devotes the whole of the contents of the book to reinforced concrete problems pure and simple. A useful idea in giving tables or graphs based on approximate formulæ is followed in also giving the percentage of error involved in using the formulæ, so that one may know how close the approximation is.

The part under review deals with reinforced concrete subject to the usual forces, axial and transverse, and combinations of both, and treats the subjects in a short and business-like manner; the most interesting chapters are those dealing with Shear and Combined Bending and Compression. The shear problem is handled in a somewhat novel manner, the author recommending the use of a shorter lever arm than is generally used. The problem is treated under three different aspects according to the way the lever arm is calculated: (1) Assuming that the concrete is able to take tension as well as compression and with the same modulus of elasticity; (2) assuming that the concrete can take tension but with a smaller modulus of elasticity than that of concrete in compression; and (3) assuming that the concrete takes no tension at all, i.e., is cracked. While for the calculation of bending stresses it has been agreed to consider the third of these assumptions as the safest to adopt, there does not appear to be any reason for following this practice in the case of shear stresses, the magnitude of which are in inverse ratio to that of the lever arm. It is shown that for all percentages of reinforcement the lever arm is shorter when taking the concrete in tension into account than if this is neglected; and as the concrete certainly can take tension, at any rate in the early stages of being stressed, the author recommends the values $0{\cdot}75\,d$ for rectangular sections and $0{\cdot}85\,d$ for **T** beams, d being the effective depth. Most other authors give these values as $0{\cdot}88\,d$ and $0{\cdot}92\,d$ respectively. In lieu of the ordinary graphs for use in connection with combined bending and compression the author has introduced a set of tables which reduces the calculation work to a minimum; the tables give the stresses in one operation, and for four different ratios of tension to compression reinforcement.

A practical example of the dimensioning and arrangement of reinforcement for a continuous beam is given, with full details of all the calculations. This, as well as all the other chapters, is illustrated by plenty of diagrams.

At the end of the book is appended a copy of the provisional Belgian Regulations for Reinforced Concrete, which apparently are the only ones existing pending the final issue of Regulations now being prepared by a Reinforced Concrete Commission.

R.N.S.

Handbook for Reinforced Concrete, Vol. III. Foundations and Walls.* 3rd Edition. By Dr. F. Emperger.

Berlin: Wilhelm Ernst und Sohn. 471 pages; 204 illustrations. Price, in England, 1,500 marks.

THIS is the third of fourteen volumes constituting Dr. Emperger's great work in reinforced concrete. In it the eminent author has had the assistance of Professors Colberg and Nowak. The subject of foundations is dealt with at great length, especially with regard to the use of reinforced concrete, illustrations and details of the foundations of several well-known German buildings being given, and, what is even more important, the various designs are duly commented upon by the author. The subject of waterproof cellars occupies a considerable number of pages, and the conclusion is reached that "there is nothing better than a 1 : 1 or 1 : 2 cement mortar, or one containing a suitable proportion of trass." When a waterproof lining is sufficient the use of a waterproofing agent is often an advantage, but the author prefers to rely on a good cement-sand mortar, carefully applied and well trowelled. In this connection the construction of concrete docks and gas-holders is well described. The foundations for various types of machines are also described, and the methods of absorption of vibration are considered.

The uses of various types of concrete "guns" for repairing defective concrete are mentioned, special reference being made to the Wolfsholz method. A description of a bridge for a torpedo-shooting school arouses many questions in the mind of a thoughtful reader.

The subject of piles occupies over one-quarter of the volume, and, with the exception that a few more or less fantastic designs of piles are omitted, is quite up-to-date, though the author refrains from specially commending any one type of pile.

The remainder of the volume deals with walls, and includes such subjects as earth pressure and water pressure, the static properties of walls, the effect of wind pressure, and the special forms of walls needed for embankments, buttresses, supporting walls, dams, etc. This section has been thoroughly revised and brought up-to-date, but does not contain anything worthy of special comment.

The volume is another example of that painstaking ability to summarise existing knowledge which has so long been a characteristic of German works on concrete and allied subjects. It contains little that is really new, and is written in a manner which largely excludes controversy. The illustrations are remarkably numerous, and most of them are very clear. In short, the book is one which no one interested in concrete construction, and able to read German, should be without.

"How to Form a Company." By Herbert W. Jordan.

Fifteenth Edition. London: Jordan & Sons, Ltd. Price 1s. 6d. net.

IN the present edition of this book there has been little occasion for alteration in the subject-matter. The book indicates the advantages that incorporation under the Companies Acts afford, and explains the machinery by which incorporation is effected. The steps that follow the incorporation of a company are shown, and the procedure of forming a company, and its subsequent management, are outlined. The book mainly deals with the formation of public companies, but the distinction between such companies and a private company is shown, and the privileges attaching to the latter class of company are set out.

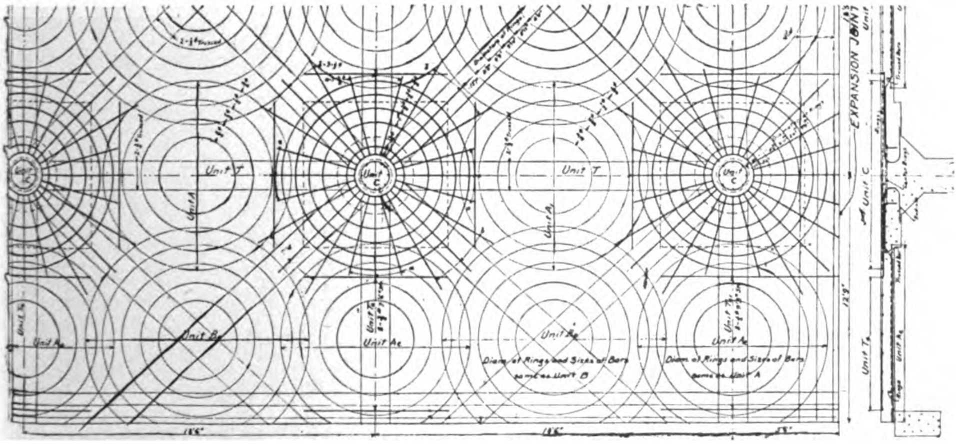
REINFORCED CONCRETE RESERVOIR IN NEW JERSEY.

THE building by the city of Perth Amboy (New Jersey) of the first 40,000,000-gallon unit of the proposed 160,000,000-gallon water reservoir has just been completed. The covered type of reservoir has been adopted. The completed unit of the reservoir is 900 ft. long, at full water level, with a width varying from 230 ft. to 370 ft., and has a water depth of 25 ft. Three sides of the reservoir consist of reinforced concrete T-shape retaining walls 7 ft. high, surmounting an embankment built from the excavated material, largely clay. The fourth side of the basin is a self-sustaining concrete wall between this unit and those to be built subsequently. This is of reinforced concrete, T-shape, 25·25 ft. high, with a 20-ft. base. The roof is a flat slab on reinforced concrete columns, and the floor slab is similar. The inside of the basin is lined with concrete. Nearly all the concrete, except the sloping lining, is reinforced. The earth embankments are carried up over the walls, and the roof is covered with 2·25 ft. of earth.

A feature of the reservoir is the design of the bottom and top slabs. Flat slab construction was used both in the floor and in the roof, the reinforcement used consisting of a combination of rings and radials. The total area of flat slab construction is about 500,000 sq. ft. The basin is provided with three construction joints which separate the floor and roof into four parts of almost equal lengths. The typical panels are 18·5 ft. sq., except that at the wall the span is shortened to 15·67 ft. to compensate for the fact that at the wall the slab is freely supported. At the expansion joints the slab is provided with a 5½ ft. overhang, designed as a cantilever, so that the spans across the joints are 11 ft. centre to centre.

The roof slab is 8 in. thick, and the drop panel at the column 7·5 ft. sq. and 4 in. thick. The 20-in. columns are reinforced with eight ¾-in. round bars and ⅜-in. hoops, and 4-ft. column-heads are used. The bottom of the reservoir is of an inverted flat slab design, computed to withstand uniformly distributed reaction. The drop panels, however, are placed below the slab and are formed in the soil. In place of column-heads, equivalent column pedestals are used. The total thickness at the column is larger than for the roof, so as to provide additional cover for the bottom reinforcement.

The reinforcement of the slabs is proportioned for a load of 300 lb. per sq. ft. The bottom reinforcement consists of units (A) between the columns



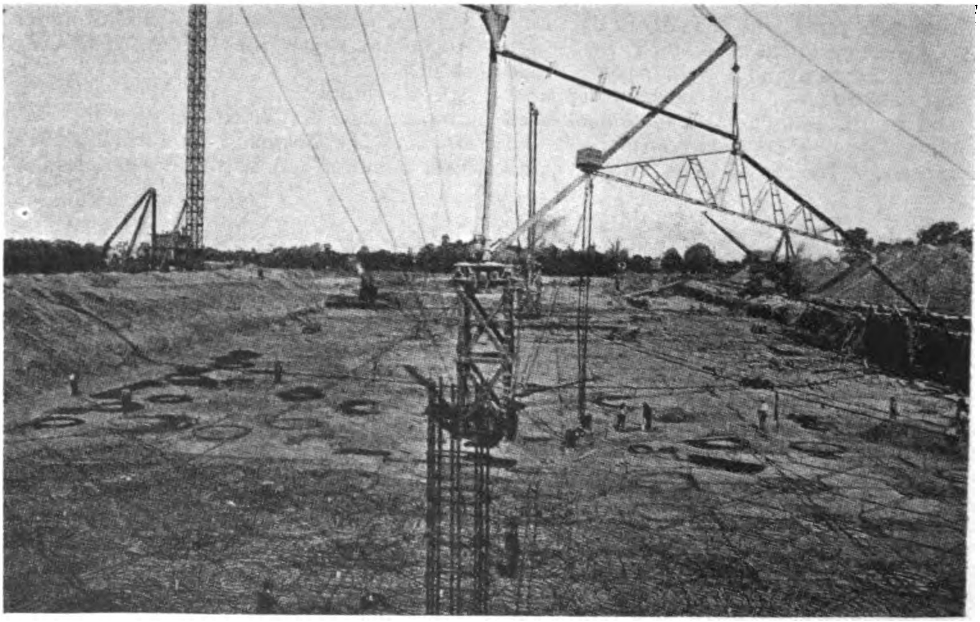
PLAN OF FLOOR REINFORCEMENT.

REINFORCED CONCRETE RESERVOIR AT NEW JERSEY.

composed of four rings ranging in diameter from 3·75 ft. to 9·75 ft., and two trussed bars carried near the top of the slab across the columns beyond the points of inflection of the adjoining panels. The central part is reinforced with six rings ranging in diameter from 3·75 ft. to 13·75 ft. (unit B), and four diagonal trussed bars, two in each diagonal direction, which are bent up in the column section and hooked to a ring placed within the column-head. The negative bending moment reinforcement at the column (unit C) consists of five rings, one of which is placed within the column-head. The diameter of the outside ring ranges from 5·5 ft. to 9·75 ft. Additional negative reinforcement is used across units "A," and consists of straight bars. The diameter of the rings is such that the various units overlap, forming in effect a continuous mat of steel. The reinforcement in the floor is placed in an inverted position, positive steel being placed near the top of the slab, and negative steel near the bottom. By the use of the rings considerable economy in steel is effected without reducing the strength of the construction.

The placing of the reinforcement was accomplished with considerable speed, although this was the first job on which the contractor had used the system. Ordinarily the trussed bars were placed first and tied at the column. This gave a good support for the rings in unit C, which were placed next. The rings in units A and B were placed last to prevent them from being disturbed by walking on them. When wired together, the steel at the column was rigid and kept its position above the form during concreting.

The formwork for the flat slab was built in towers, one tower for each panel. The portion between the columns was hinged in such a way that forward movement of the towers was possible. Sufficient formwork for one section was provided; concreting was started near the adjoining section, and as soon as the concrete had hardened sufficiently the forms were advanced to the other section. The towers were thus used four times.



Floor with Portion of Reinforcement in Place, Ready for Concreting.

REINFORCED CONCRETE RESERVOIR IN NEW JERSEY.

REINFORCEMENT FOR SHEAR IN REINFORCED CONCRETE BEAMS.

At a meeting of the Institution of Structural Engineers held on February 22, Mr. Edward Godfrey, M.Am.Soc.C.E., M.I.Struct.E., read a paper entitled "Reinforcement for Shear in Reinforced Concrete Beams," in the course of which he said :—

In every beam there are three principal elements that must be taken care of in the design before the beam can be considered properly designed or safe. One is provision for tension in the bottom part of the beam in the portion where the bending moment is the maximum (assuming a positive bending moment). In a reinforced concrete beam this is effected by the steel rods which must lie near the bottom of the beam where moments are high and must extend to the ends for grip or anchorage. The second element is provision for compression in the upper part of the beam. This is taken care of by supplying width and depth of beam of sufficient magnitude to keep the unit stress on the extreme upper edge within safe limits. The third requisite is provision to take the shear in every part of the beam, particularly that to take the end shear to the support. It is in provision for the shear, or lack of provision for it, where the great errors in design are made and where the cause of many failures is located.

Analysis of shear and of methods of shear reinforcement in concrete beams is one of the most illogical things in all the literature on this subject. Standard methods of taking care of shear in reinforced concrete beams are uniformly illogical and inefficient. The truss idea of steel construction has been adopted by the reinforced concrete designer and authority, but they have failed to observe that their truss web members, the stirrups and short shear members in reinforced concrete beams, lack vital essentials that must be in every truss. Another false line ot reasoning employed by advocates of stirrups is analogy with built-up wooden beams—stirrups being assumed to act as pins or bolts.

It is proposed to show that the analogy which asserts that a reinforced concrete beam is a modified Howe or Pratt truss is totally false, and that the one that asserts that stirrups act in a concrete beam as bolts or pins is also totally false. As the standards of designing for shear in reinforced concrete are based on these two ideas, these standards are on a false basis and hence should be discarded. Still another line of reasoning applied to the designing of shear reinforcement is a theoretical one that attempts to discover the intensity of shear in the concrete for the purpose of spacing the stirrups. This is also a blind type of reasoning that falls short of any conclusion as to what becomes of the stress in the stirrup or how it can meet any adequate resisting medium. In fact, all the three lines of reasoning referred to have this common fault, namely, they apparently discover a stress in a stirrup or short shear member but they fail utterly to follow out that stress to an adequate reaction. It is exactly as though one should discover certain stresses in the web members of a truss and section enough to carry those direct stresses, and on this bare premise declare the truss to be safe, ignoring altogether the fact that the end connections of these members are totally inadequate to take the stresses.

After citing a number of references to substantiate his assertions as to the nature of standard analyses for shear and standard methods of reinforcing for shear, the lecturer continued :—

I have made a number of tests on large buildings where the shear reinforcement was by means of main rods bent up at angles less than 20 deg. and anchored over the supports. The results were excellent, and no cracks were found in the beams. I have also tested other floors of large buildings where the shear reinforcement was vertical stirrups. The results were exceedingly bad, and cracks in the beams were very numerous, and some of them very wide open.

To sum up the situation as regards shear or diagonal tension reinforcement and analysis of the same :

(*a*) There are engineers and recognized authorities who hold that steel rods embedded in concrete can take shear at units such as 10,000 to 12,000 lb. per sq. in.

(*b*) There seem to be no standard

specifications that allow such shear, or any stated shear units, on steel rods embedded in concrete, though such were common fifteen years ago.

(*c*) There are engineers and authorities who say that no such allowances can be made on steel in concrete. There are those who aver that this has been entirely abandoned.

(*d*) Engineers and standard authorities are almost a unit in their adoption of the following three methods of reinforcing a beam for shear or diagonal tension: (1) Vertical stirrups; (2) Stirrups inclined at about 45 deg. with the horizontal; (3) Bent-up ends of main rods, bent up at angles of about 45 deg., the rods ending in the upper part of the beam. All these methods are by short shear members. The shear members are inclined at a large angle with the horizontal. Whether they are connected by means of loops or by being part of the horizontal bottom rods is a detail that does not affect the principle very much. The thing that stands out in all standard practice in reinforced concrete is that shear is to be provided against by short shear members, and only rarely or recently are those members required to have any semblance of anchorage into the supports. Some authorities have recently got under cover in this matter.

(*e*) Many designers and some authors, seeming to sense the futility of disjointed short rods as a reinforcement for shear, throw in for safety main rods bent up with a small angle and anchored over the supports. Very recently this type of shear reinforcement, without stirrups, has been included with other types in the regulations of the American Concrete Institute and in the Joint Committee Report.

(*f*) Tests prove that in a whole beam stirrups and bent-up rods of the standard type receive no stress whatever of any consequence. They may even receive a small compressive stress, which is the very thing that they are not designed to do. Tests also show that when the beam is broken up by shear or diagonal tension cracks in the concrete, the short shear members are, of course, subject to tension.

(*g*) Tests have proved that when a beam is reinforced for shear or diagonal tension by bending up main tension rods to the top of the beam at supports and anchoring these rods beyond the edge of support, the bent-up portion of the rod being at an angle of 20 deg. or less with the horizontal, excellent results are obtained. The rods are found to take large stress while the beams are whole, thus proving that they are doing the work expected of them, and the beams are frequently not even cracked under high loads. And this is the type of reinforcement not even permitted by the latest American standard (the Joint Committee Report), though that report does allow, in its latest edition, a steeper angle to reinforcing rods than 20 deg. Standard works in America and England make no mention of this method of reinforcing for shear, except that they hint that it may be used as an extra precaution. Before going into detail in the matter of further demonstrating the error of the stirrup, I shall illustrate and explain true reinforcement for the shear of a reinforced concrete beam.

Figs. 1 and 2 show rational and economical reinforcement for the shear of a beam. By the shear of a beam is meant the sum of the vertical forces which on one side of a vertical section are upward and on the other side of that section are downward. It matters little whether in the concrete itself the stress be interpreted as shear or diagonal tension, the big thing is to provide reinforcement that will safely and economically resist the shear of the beam or at least that part of it which is beyond the capacity of the concrete itself to carry. That the type of reinforcement shown in *Figs.* 1 and 2 is economical is beyond dispute. That it saves a lot of time and trouble in the field as against a mass of small stirrups or shear members is equally evident. That it is a type of reinforcement that can be completely analysed and fulfils every condition of rationality has never been questioned. That it has proven itself to be the surest and best

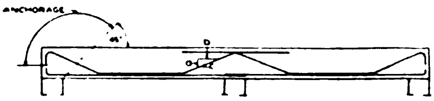


FIG. 1.

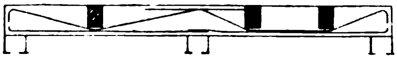


FIG. 2.

reinforcement against beam failures due to high shear is a matter of record proven by tests made by men who never publicly recognised it as even an alternative method of reinforcement until the most recent time.

The elements of this reinforcement are very simple. In the first place a beam that has the main steel anchored into the supports both at top and bottom of the beam is most favourably conditioned as regards the ability of the concrete itself to take shear: for even if the concrete should crack, the roughness of the surfaces, and the reinforcing rods which hold these surfaces firmly together, would enable the beam to carry a large shear. It is therefore admissible to allow a good shear unit on the concrete. A unit shear of 60 lb. per sq. in., or even more, can safely be allowed on the full section of the rectangle of the beam. The shear of the beam in excess of this amount should be taken as a direct stress in the inclined portion of the main steel rods, which are bent up and anchored beyond the edge of the support. To find the stress in the bent-up rod or rods multiply the excess of shear over that carried by the concrete by *c* in the small triangle of *Fig.* 1 and divide by *a*.

In a beam carrying uniform load, the rods should be bent up at about quarter points in the span. They should, of course, be bent up at points not nearer to the support than the section of the beam where the shear equals sixty times the area of the cross section. This will generally give or allow a small angle with the horizontal. There has never been given any published reason why this angle should exceed 20 deg., as required by the Joint Committee Report, except the publicly stated personal view of the man who put the defenceless limitation in the Report. It is a fact that the report submitted to the American Concrete Institute in 1920 prohibited any reinforcement for shear that was bent up at a smaller angle than 30 deg. with the horizontal. This was changed before the report was adopted, because of my urgent protest.

In beams carrying concentrated loads, as in *Fig.* 2, the rods should be bent under the beams, or approximately so. The bent-up rods should be anchored into end supports as indicated in *Fig.* 1, or by some other equally efficient method. If the length of rod beyond the 45-deg. plane is about forty or fifty diameters, the anchorage should be sufficient.

Turning now to the stirrup or short shear member, and the arguments against the same, the first argument is a set of figures showing what for many years has been considered standard and safe design for shear reinforcement of beams and showing the lines where these beams can fail *by shear* without disturbing the alleged shear reinforcement. *Figs.* 3, 4 and 5 are these figures. These figures have been published before as an argument against short shear rods.

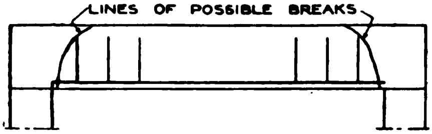


Fig. 3.

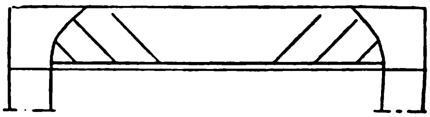


Fig. 4.

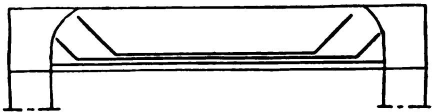


Fig. 5.

Formerly there was no demand for stirrups to be anchored either by embedment in the concrete or by attachment to the horizontal steel. Now there is some recognition of the benefit of tying the beam together by anchoring and connecting the reinforcing steel. The authorities referred to in the early part of this paper call for anchorage in the following ways:

(1) Some say that stirrups should have an embedment in concrete that will develop their full strength above the neutral axis.

(2) Some say that it is sufficient if in the full length of the stirrup the strength is developed, one end being looped around the bottom steel for the reaction.

(3) The Joint Committee in its latest Report calls for looping around horizontal steel at bottom and at top of the beam in cases where the unit shear is exceedingly high.

All of these authorities must, of necessity, use the truss analysis to account for the stress in the stirrup. No other analysis has ever been set up.

Let us examine these methods of anchoring stirrups in an effort to answer the question, "What becomes of the stress in a stirrup after it leaves the stirrup?"

Take a stirrup that is looped around a horizontal rod, say the stirrup is a ½-in. sq. rod. The alleged stress is 8,000 lb., a vertical force. What becomes of it? It is a force that must reach the support of the beam. Is it taken by shear in the horizontal rod? This is impossible and absurd. And yet there is no other medium but the rod, and the concrete which is supposed to be relieved of shear by the stirrup, and absolutely no way in which it could be taken except by shear in that rod.

Next, consider the stirrup which requires its full length for embedment to effect anchorage. How can this be considered a vertical member? It has its full stress in a length of zero.

Next consider the case of the stirrup that has its full anchorage above the neutral axis. What about the stress or the shear of the beam above the neutral axis? How can a stirrup be considered as a vertical member in a truss if its effective length is only one half of the depth of the truss?

The case of the stirrup that is looped around horizontal rods at top and bottom of the beam is again that of a right angle pull against a rod which cannot be taken in any other way than by shear in the horizontal rod. Besides this absurdity the upper horizontal rods are put there to take tension. How can the vertical pull of the stirrup be converted into horizontal tension in the top rod? No advocate of stirrups has ever attempted to reply to this sober and searching analysis of the stirrup. The same analysis applies, with a little variation, to all short shear members—to all systems that employ sharp bends in steel rods in tension. The right angle bend (the vertical stirrup) or the sharply inclined bar (diagonal stirrup) cannot be defended.

One of the strongest proofs of the error of the stirrup is the character of the arguments made in defence of the stirrup and in a large measure the avoidance of the topic by men who hold up the stirrup as a proper type of reinforcement.

In a beam that is cracked by reason of high shear, vertical stirrups will be found to be in tension, otherwise the beam would be almost sure to collapse. Stirrups, if there are enough of them, will delay the failure, or, if stirrups are lavished on the beam, the stirrups will prevent failure. Tests are not needed to demonstrate this; simple analysis is sufficient. If, for example, initial failure, as exhibited by the cracked beam, throws the load of the beam on the horizontal rod in shear or on insufficiently anchored stirrups and then on the concrete between stirrups, as a shearing stress, the system is at fault, and some more rational method of reinforcing a beam for its shear should be sought. To prove that in a whole beam of concrete of a uniform character there will be little or no stress of any kind in the stirrups, it is only necessary, by analysis, to ask: What possible condition can there be which would produce vertical elongation or vertical shortening in the beam? This question answers itself. If the beam cannot be elongated or shortened vertically, it is quite impossible that the embedded stirrup could be subjected to tension or compression.

It is quite easy to show by analysis that if there are layers of weak or mushy concrete in certain strata of the beam there will be tension or compression in the stirrup, depending on the way in which these layers of "mush" may lie. *Fig.* 6 illustrates this. At the left end of the beam stirrup A will be in compression, and it will be compelled to take the full shear of the beam, if the layer of "mush" is inclined as indicated; not half the shear, as standard analysis has it. At the right end of the beam stirrup A′, identically conditioned with A, will be in tension, the full shear of the beam.

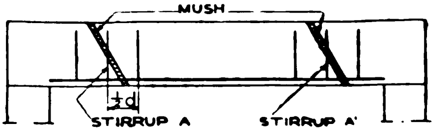


FIG. 6.

Tests have found little stress of any kind, in stirrups in whole beams. Some tests on a building did discover a trifling amount of compression in stirrups. Speaking of these tests, I have made the statement "that tests have failed to find any measurable tension in a loaded beam." This statement that tests have failed to find tension, where manifestly tension was sought, has been seized upon with avidity by proponents of the stirrup as being one of the false statements that I am so frequently accused of making. The tests which found a small compression in stirrups and therefore "failed to find tension" are those already referred to in this paper in Bulletin No. 64, University of Illinois Experiment Station.

As compared with the freak tests made on beams where stirrups are so close and numerous that they would give a contractor the nightmare if used in actual construction, the number of tests made on beams where there are no stirrups, but where main tension rods are bent up with a flat bend and anchored, have been few. Some of these have had the rods very imperfectly anchored and have failed on that account. Some early tests show poor results because of the dry concrete used in those days. Steel is not properly gripped in dry concrete.

This paper would not be complete without a reference to the structural principles that are at once an explanation for the confidence on the part of designers in short shear members and the very general interpretation by authorities of the results of tests on stirruped beams as warrant for that confidence, for, of course, there must be some reason for a stand that is held to with such tenacity by a whole profession. Reinforced concrete authorities have very recently recognised the benefit of end anchorage of main steel reinforcement by allowing twice as much end shear in a beam so conditioned. It can be shown that a beam is better capable of resisting its own end shear if the main reinforcing rods are anchored over the supports, though there be no shear reinforcement. It can also be shown that this beam shear is not taken by shear in the concrete but by compression, a stress which concrete is well able to take.

Given a simple truss, as in *Fig.* 7, composed of two top chord members and a tie, roughly the chord stress is 100,000 lb. If the top chord has 100 sq. in. of area, the unit stress is about 1,000 lb. On the 100 sq. in. of cross section of the compression chord the apparent vertical shear in the chord is about 100 lb. per sq. in., or approximately one-tenth of the intensity of the compression. It is clear that there is not a vertical shear in the chord, but a simple and direct compression. The shear of the truss is taken by the vertical component of that compression. But suppose this were a concrete beam, the triangular portions being filled out, as in *Fig.* 8. It could be said that the beam has no vertical shear provided one thing were true, that is, that the reinforcing rod had a full anchorage beyond the point of support. It is just here that much false interpretation of tests exists.

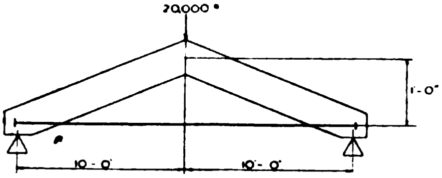


FIG. 7.

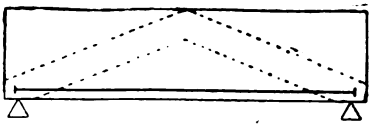


FIG. 8.

The truss idea may be varied as in *Fig.* 9. If in this figure the steel rod has full anchorage beyond or at the point of support, there is no shear whatever in the concrete member, not even as a vertical component of direct stress. The whole shear of the truss is taken by the steel rod as a vertical component of the stress in the inclined portion of the rod. By adding concrete to complete the beam, a reinforced concrete beam, as in *Fig.* 10, could be constructed that could be considered to have no shear in the concrete, provided the steel rods were capable of carrying the calculated stress,

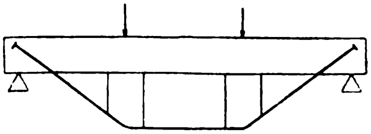


FIG. 9.

and provided, of course, that the ends had full anchorage at points over the supports. (It is not attempted here to show the manner of anchorage, but merely to indicate that the rods have some form of full anchorage beyond the points of support.)

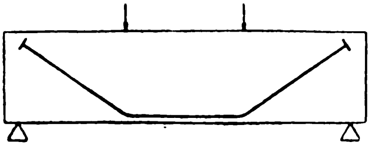


FIG. 10.

Tests have been made and reported that purport to show exceedingly high stress in stirrups, such stresses as 100,000 lb. per sq. in., stresses that are totally fictitious, and that are based on an imaginary distribution of shear. The fact is that in a large measure these stirrups were functioning as lateral reinforcement in a compression member inclined a little to the horizontal. For in these laboratory tests the bottom rods always extend beyond the edge of support, and thus approach the condition of the rods of *Fig.* 8; and, besides this, the supports themselves in laboratory tests are generally rigid blocks, which offer resistance to horizontal displacement. These two features of laboratory tests (partial end anchorage of horizontal rods and horizontal resistance of supports), as causing arch action, have been ignored by experimenters, and hence interpretation of results as pertaining to shear are of little value.

The reasons why end anchorage of the horizontal tension rods is not as beneficial in reinforcing a beam for shear as bending up and anchoring some of the rods, are the following :—

(1) In the beam indicated in *Fig.* 8 the shear of the beam is carried by diagonal compression in concrete. In the one indicated in *Fig.* 10 the shear of the beam is carried by tension in the steel rod. Tension in a steel rod is more to be depended upon than compression in concrete. Steel is made under supervision; concrete is not always so made.

(2) Most beams in buildings are T-beams, and the facilities for anchoring steel rods in the stem of the T are not good; they are not to be compared with those in the upper part of the beam.

(3) The inclined compression member of *Fig.* 8 in a T-beam will be very much constricted near the ends of span, as only the stem of the T. is available.

(4) On the other hand, the horizontal compression member of *Fig.* 10 will be of excellent shape for a compression member, not only because the flange of the T is added, but also because the floor slab adds its stiffening effect.

(5) In continuous beams, which are more common than other types, the bent-up rods shown typically in *Fig.* 10 would in reality extend into the next beam for top tension reinforcement, as shown in *Figs.* 1 and 2.

It is manifest from the foregoing analysis that bending up rods into the top of beams with a sharp bend, in any of the standard ways of reinforcing for shear, does not constitute an efficient shear reinforcement. This analysis further makes it clear why a large number of closely-spaced stirrups will add shearing strength to a beam, if the bottom tension is taken care of by rods anchored beyond the edge of support. The shear of the beam is broken up into zigzag lines of tension in steel rods and compression in nearly vertical lines in the concrete. If the rods are widely spaced, as in standard design, and if the bottom tension is not taken care of by anchored rods, the beam may break between stirrups and failure may result, even though the design may be in every respect exactly what is required by standard regulations. Hence it is a crime to design a reinforced concrete beam which depends upon stirrups or short shear members to carry the end shear.

DISCUSSION.

Dr. **OSCAR FABER**, opening the discussion, said there was a great deal in the paper which ought not to be accepted. It was difficult sometimes to make out clearly where the lecturer differed from orthodox practice, and what he was getting at. Referring to the remark that "the truss idea of steel construction had been adopted by the reinforced concrete designer and authority, but they had failed to observe that their truss web members, the stirrups and short shear members in reinforced concrete beams, lacked vital essentials," he (the speaker) said the only vital essential they sometimes lacked was end connection, and that only in bad design. Obviously, if they had a truss in which the members were not connected at their ends by a bolt or other efficient connection, that truss would be weak; if they had short shear members not adequately connected at the top and bottom to the tension and compression member respectively, the truss would not act. There was no reason, however, why a truss or short shear members should not be connected adequately at their ends to the compression or tension member respectively. Later, the lecturer had said he proposed to show that the analogy which asserted that a reinforced concrete beam was a modified Howe or Pratt truss was totally false, and that the one that asserted that stirrups acted in a concrete beam as bolts or pins was also totally false. He (the speaker), however, did not think the lecturer had shown that. On the contrary, towards the end of his paper he had shown diagrams and arguments based entirely on the truss idea. The lecturer had shown that a great many beams which had stirrups which were not connected at their ends did not act as a truss, but the speaker could find nothing in the paper which showed that a beam with stirrups properly anchored at their ends did not behave as a proper Howe or Pratt truss. In a criticism of the long list of references to the works of various authorities which were given in the paper to substantiate the author's assertions as to the nature of standard analyses for shear and standard methods of reinforcing for shear, the speaker pointed to a reference to a book written by himself and Bowie, in 1912, on "Reinforced Concrete Design." There were several things which, according to the lecturer, he was supposed to believe, but which, as a matter of fact, he did not, and if that was as well as the lecturer could interpret his expressed opinions on the subject of shear, he was wondering whether Mr. Godfrey had got the expressed opinions of other people in the list any more accurately. Further, although the lecturer had referred to that book, which was written in 1912, he had ignored the paper which the speaker had read before the Concrete Institute in 1916, on shear resistance in reinforced concrete beams, which contained the results of three or four years' experimental work carried out subsequently. Dealing with the paper broadly, it appeared to him that the lecturer's view was that they could not treat shear reinforcement in a beam by considering it as a truss; if that were so, he entirely disagreed. The only way to get a rational understanding of the action of the various components in a beam was by considering them as a truss, and if they did that they would get brought prominently before their notice what were the factors which would limit the strength of that truss. They had to consider every compression and every tension member, and see that the concrete was strong enough everywhere to take the compression, and the steel strong enough everywhere to take the tension; to consider the connections or joints between the compression and tension members; to consider the adhesion of the steel members in the concrete; and, more important still, to see that the bearing stress on the diagonal members against the bends in the bend-up bars was not excessive. There was no other way of arriving at what a particular beam would carry in shear. The second main point of the paper was that short stirrups and short shear members generally were no good, but he had given no very convincing figures to prove that. He (the speaker) presumed that the lecturer had not seen the paper he (Dr. Faber) had read before the Concrete Institute in 1916, already referred to, but in that there was fairly conclusive evidence as to the value of stirrups and the proper method of calculation. His calculations were fully explained in that paper.

Mr. Ewart S. Andrews said one of the things upon which he believed all British designers agreed was that if they were going to design shear reinforcement it must be designed to carry the whole of the shearing forces, but American designers did not appear to follow that doctrine. In the American standard regulations, and in American text-books, they provided only sufficient steel to reinforce against the shear which the concrete could not carry, and we in this country were wise in not following that practice. Therefore, the lecturer was trying to pull down a bogey which reinforced concrete designers in this country had pulled down long ago. After a reference to the various phases through which the stresses in a reinforced concrete beam passed, Mr. Andrews said it seemed to him that one thing that should be impressed upon the younger men was that shear reinforcement was not shear reinforcement at all, but only another kind of tensile reinforcement, i.e. the reinforcement to carry the tensile components of what we called the shear. If they bore that in mind they would not expect designers to follow the methods which the lecturer appeared to condemn. Much as they might criticise the regulations of the London County Council, he did not believe we could design a beam in which the shear reinforcement would fulfil the requirements of those regulations, but which, at the same time, would be obviously faulty in shear.

Mr. G. A. Gardner said the paper was critical rather than convincingly constructive, and he thought its value would have been enhanced had the mention of various tests been supplemented by the necessary data and records of measurement.

Mr. M. E. Yeatman said the rules which

were put forward in the paper were exactly those which he had always followed. He had never found a simple rectangular beam in which stirrups were needed, but in T-beams the case was different, and stirrups might be needed. A point on which he did not agree was in regard to his statement that rods should be bent up with a flat bend and not a sharp bend; he gathered the author meant a small angle from the horizontal. He did not think it made any difference whether the angle were small or large so long as the bend was a gradual curve and not a sharp one.

Mr. W. A. **Green** said although he had designed beams with short stirrups he had taken good care to anchor them over a horizontal rod. He agreed with the lecturer with regard to shear stress in the rods over which the web members or vertical stirrups were anchored, but he did not think that was any disadvanatage.

Dr. **Faber**, dealing with the point as to whether there was proof that there was tension in stirrups, said that when reading his paper before the Concrete Institute, to which he had referred, he had shown photographs of beams which had failed in shear. Those beams were reinforced with stirrups, and when they failed they showed horizontal cracks about half-way up the beams, which clearly indicated that there was tension on the vertical stirrups.

Mr. H. **Kempton Dyson** said he agreed with all that Dr. Faber had said, and with the method of analysis which Dr. Faber had described in his paper in 1916. The only thing that might be said to supplement Dr. Faber's criticism, or rather to point to one other aspect of it, was that the lecturer seemed to assert that vertical stirrups, though they might serve to produce truss action when the beam was cracked, were really useless, because they only came into play when a beam was cracked. He had seen a large number of beams in structures tested in practice, but had not seen them without cracks, and they could not develop 16,000 lb. per sq. in. in the bottom tension bars without getting cracks in the concrete. Every test published in America had shown cracks at an early stage, and the girders referred to by Dr. Faber in his paper showed cracks, bringing the stirrups into play, and his view was that they did produce truss action. Dealing with the angle of bent-up bars, he had designed a number of beams which had been subjected to very heavy shear, in which the main bars were bent up 45 deg. That was a particularly economical mode of reinforcement, rather than to incline bars at 15 or 20 deg. as the lecturer suggested.

CONCRETE PRESBYTERY AT GRIFFITH, N.S.W.

Below we illustrate a Roman Catholic presbytery recently built in concrete at Griffith, New South Wales. The presbytery is built in mass concrete on sandy loam soil on a foundation 1 ft. deep; the concrete used for the structure was in the proportion of 1 part Portland cement to 8 parts of sand and coarse aggregate.

Presbytery (built in Concrete) at Griffith, N.S.W.

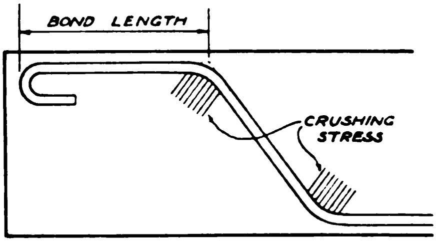
QUESTIONS AND ANSWERS RELATING TO CONCRETE.

Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.—Ed.

RESISTANCE OF BENT-UP BARS.

Question.—Regarding the shear resistance of bent-up bars and stirrups, I note that Dr. Oscar Faber, in your issues for February and April, 1922, recommends a stress in the steel not exceeding 10,000 lb. per sq. in. Is there any reason why the usual stress of 16,000 lb. per sq. in. cannot be used, provided that (*a*) all the shear is taken by the bent-up bars and stirrups, and (*b*) that the bent-up bars and stirrups are given adequate bond length? As far as I am able to see, the L.C.C. Regulations regarding shear permit a stress of 16,000 lb. per sq. in. in the steel. Regarding (*b*), from what point in the bent-up bar or stirrup should the bond length be estimated? Hood and Johnson recommend 0·6 of the effective depth of the beam.—T.C.G.

Answer.—If precautions (*a*) and (*b*) are taken, it still remains necessary to see that the bearing stress on the concrete at bends is not excessive, so as to crush or split the concrete. Even if the bars are bent to as large a radius as is practicable in ordinary cases, this stress will still be found to be grossly excessive in most practical cases unless a low stress is taken in the shear reinforcement. In our view, the bond length should be measured from the bend, as shown, but the total force less the resistance of the bend only need be provided for.—O.F.



CORRESPONDENCE.

COLLABORATION BETWEEN ARCHITECT AND ENGINEER.

Sir,—In December, 1921, I ventured to draw the attention of your readers to the desirability of collaboration between architects, borough engineers, etc., and structural engineers and specialist designers in reinforced concrete. The matter was discussed at the Institution of Structural Engineers on January 25 this year, when a paper by Mr. W. J. H. Leverton was read. The statement was then made that it would be undoubtedly to the building owner's advantage if collaboration took place at the early stages.

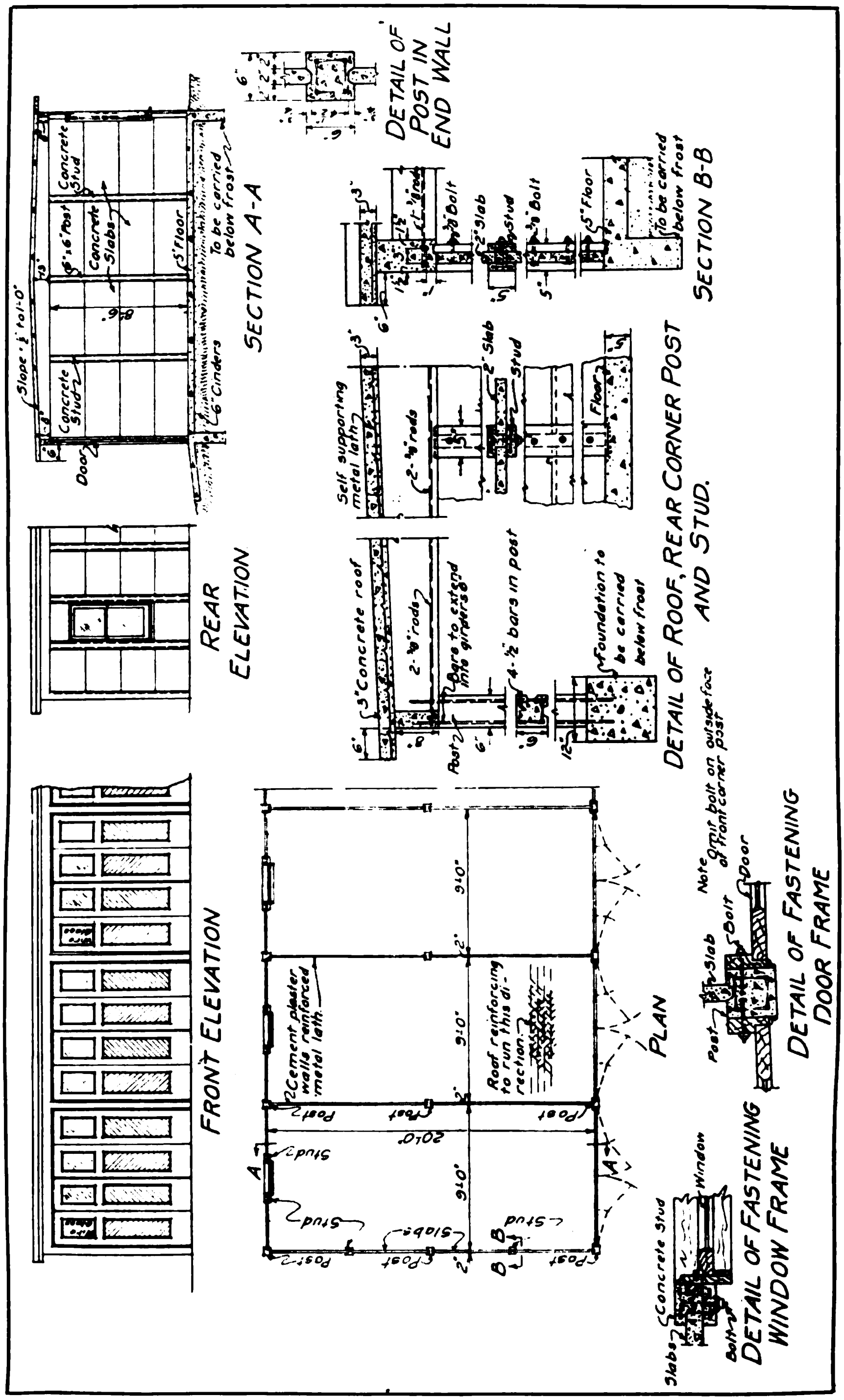
The unnecessary cost of producing many designs, which cost must ultimately be borne by the building or structure owner, is very well illustrated by the publication in *The Builder* for March 9 of the tenders received by the Borough of Crewe for Olston Street Bridge. Forty-two estimates were sent in, based upon thirteen different designs. The estimates on one particular design varied from £4,018 2*s.* 5*d.* up to £7,205 8*s.* 8*d.* The actual cost of preparing those thirteen designs would be more than equal to half the cost of the whole work.

Another case is known to me where no fewer than one-hundred-and-sixty different estimates were obtained with a proportionate number of separate designs.

It has been urged on national and other grounds that economy is absolutely essential if progress is to be made. I wonder if any of your readers can point out where economy is effected by such procedure? Even the Manchester School of Economics, I think, would shudder at such waste of effort.

Harry Jackson.

Birmingham.


FRONT ELEVATION
REAR ELEVATION
SECTION A-A
Slope ¼" to 1'-0"
Concrete Stud
Concrete Slabs
5"×6" Post
8'-6"
6" Cinders
5" Floor
To be carried below frost
Door
DETAIL OF POST IN END WALL
6"
SECTION B-B
⅜" Bolt
2" Slab
Stud
5" Floor
To be carried below frost
DETAIL OF ROOF, REAR CORNER POST AND STUD.
3" Concrete roof
2-⅜" rods
Bars to extend into girders 8"
4-½" bars in post
Foundation to be carried below frost
Self supporting metal lath
2" Slab
Stud
Floor
Post
PLAN
Cement plaster walls reinforced metal lath.
Roof reinforcing to run this direction.
9'0"
2'
20'0"
Post
Stud
Slabs
A
B
DETAIL OF FASTENING DOOR FRAME
Note: Omit bolt on outside face of front corner post
Slab
Bolt
Door
Post
DETAIL OF FASTENING WINDOW FRAME
Slabs
Concrete Stud
Window
Bolt

Design of Pre-cast Concrete Multiple-Car Garage. (See p. 245.)

PORTABLE CONCRETE GARAGES.

With the rapid increase in the number of owners of motor-cars and motor-cycles there is a considerable demand for a garage which is easy to erect, portable, cheap both in first cost and maintenance, and which will have a longer life than a cheap wooden structure.

The garages illustrated in the photograph, with the exception of the concrete floor, which is cast on the ground, are constructed of pre-cast units, including slabs, pilasters, sills, plates, collar beams, rafters and other necessary pieces. The units are kept to such a size that two men can easily handle the largest pieces.

In building the garage the concrete floor is placed first. After this has hardened a pilaster 8 ft. high is erected at the corner and a section of concrete sill placed. Then one of the slabs, which are 33 ins. wide, 8 ft. long and 1 in. thick, is put into place. The pilasters and sills are slotted so that the edges of the slabs fit into them. Next, an intermediate pilaster, slotted on two sides, is erected; another section of sill is laid, and the second slab placed. This operation is continued until the wall is completed. Pre-cast plates similar to the sills are then fitted on the top of the wall. To prevent spreading, opposite sides of the wall are tied together with pre-cast collar beams. These beams are bolted at the ends to the pilasters by means of bolts embedded in the pilasters at the time of manufacture. The roof, which is of slab construction, is supported by .pre-cast concrete rafters 3 ft. apart. The lower ends of the rafters rest on the pilasters and are bolted to them with the same bolt that passes through the collar beams.

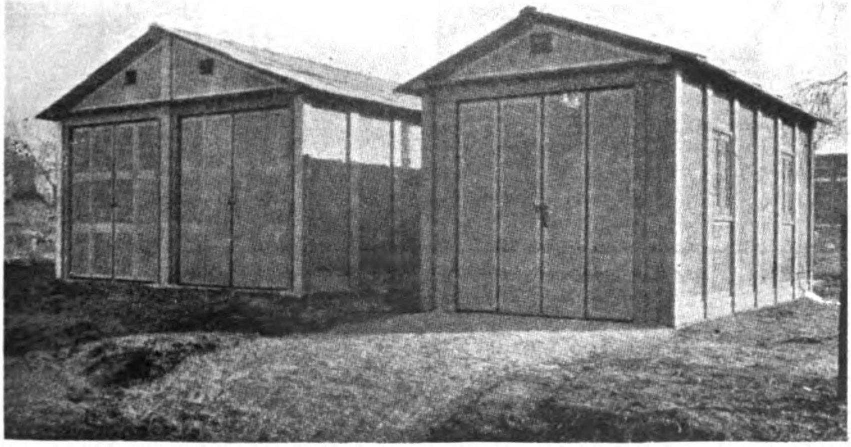
The doors consist of concrete slabs cast in steel frames, which are provided with hinges so that the various sections will fold together. Bolts for the attachment of hinges are embedded in the concrete corner piers when manufactured.

Window sashes are made of concrete.

No mortar is used anywhere in the construction of the garage, and if at any time it is decided to move the building it can be taken apart readily and the units transported to the new site and rebuilt.

The concrete units for such a building would be manufactured in bulk in a concrete products factory and delivered in sections for erection on the site. In addition to their use as garages, this type of building should be in demand for a variety of purposes for which a cheap, portable, durable, and fire-resisting structure is required.

On p. 244 are given detail drawings for a larger garage built of pre-cast units.

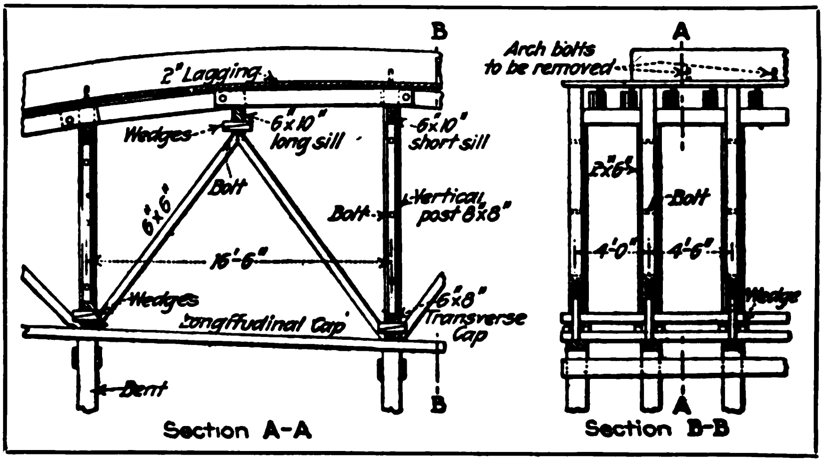


Portable Concrete Garages.

SEMI-REMOVABLE CENTRES FOR CONCRETE BRIDGE ARCHES.

In a recent issue of *Engineering News Record,* Mr. Howard G. Peterson describes the method adopted in the construction of the reinforced concrete bridge at Black Street, Hamilton, Ohio, where the necessity for keeping falsework under the completed arches until all seven were poured led to the design of semi-removable centres. The plan was to remove the lagging, stringers, caps, and the batter posts, shortly after the concrete had set. The undisturbed skeleton structure which remained then took the vertical load from the green arches and also supported the spandrel walls and cantilever sidewalks. The parts which were removed were re-used on the new arches.

The arches are each 38 ft. wide and 93 ft. centre to centre of piers. Five bents, placed about 16 ft. apart to permit flood waters to pass, supported each arch. A deck of longitudinal timber was placed on top of the bents on a line with the tops of the piers. This deck helped to brace the bents and also took the side thrust from the batter posts. Caps were placed on top of the deck, one over each bent, which supported the wedges for the vertical posts and also acted as heels for the batter posts. The vertical posts were placed 4½ ft. on centres and extended to within about 2 in. of the intrados of the arch. Pieces of lagging were placed over them and provided with bolts to fasten them to the arch when it was poured. Their purpose was to brace the posts after the stringers and much of the bracing had been removed. A strip of 2 in. by 6 in. lumber was bolted to the side of each post, and carried short caps between the posts, which in turn carried the stringers. When the stringers were to be removed the strips were unfastened and sprung out of plumb, thus lowering the short caps. The batter posts were placed entirely independently of the vertical posts and were provided with wedges at the top. The stringers and lagging were placed in the ordinary way. Only three sets of the removable parts were used.



Semi-Removable Centres for Black Street Bridge, Hamilton, Ohio.

New Pumping Station at Aberdeen.

The British Steel Piling Company, of Dock House, Billiter Street, London, E.C.3, ask us to state that the damaged and distorted piling, illustrated in the article on the new pumping station at Aberdeen Electricity Works in our last issue, was not supplied by them.

NEW REINFORCED CONCRETE BRIDGE AT LOUGHOR.

THIS bridge (illustrated on p. 248) is being built at the joint cost of the County Councils of Glamorgan and Carmarthenshire, in substitution for, and on the site of, the old timber trestle structure, which was built about the year 1834. It spans the tidal estuary of the River Loughor, and carries the Swansea and Llanelly main road, which is an important section of the main route from London to Milford.

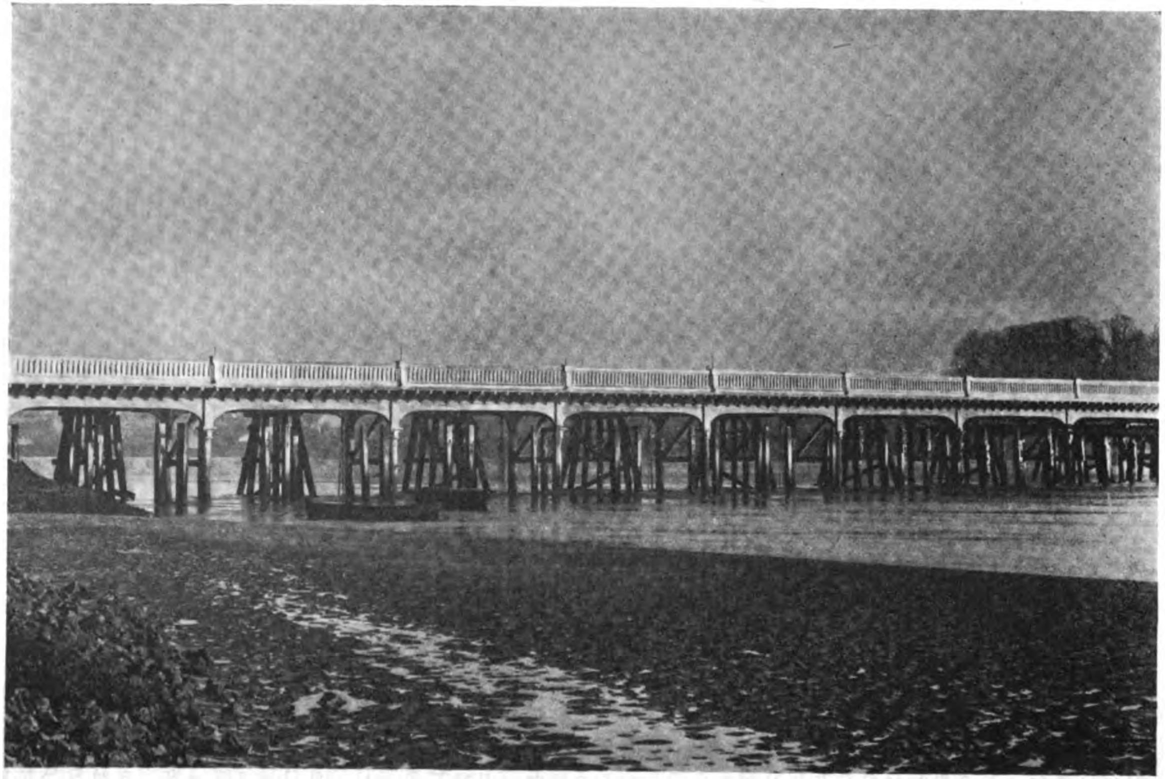
The following particulars and description are taken from a paper recently read before the Institution of Municipal and County Engineers by Mr. Geo. A. Phillips, M.Inst.C.E., County Surveyor of Glamorgan.

Competitive designs and estimates were publicly invited, and several schemes were submitted for steel structures, some with stone piers, and also for reinforced concrete structures. After consideration it was decided to construct a reinforced concrete bridge. The estimates for the schemes submitted naturally varied with the material proposed to be used, but generally the steel structures were the more costly. The question of first cost did not so much influence the decision in favour of the concrete structure as the question of future maintenance. The cost of maintaining a reinforced concrete structure was considered to be practically negligible, while the constant cost of painting a big steel structure situated in such an exposed position, and the prevailing moist atmosphere, would amount to a very serious annual charge, estimated at that time (pre-war) to amount to at least £130 per annum. At the present time it would probably cost twice as much.

The scheme was delayed by the war, but after the war the great increase in the amount, and the change in the character, of the traffic on the highways made the necessity for proceeding with the work very much more urgent. The Joint Committee of the two County Councils invited tenders for the construction of the bridge in accordance with a modified design submitted by Messrs. L. G. Mouchel & Partners, and a contract was entered into with the Yorkshire Hennebique Contracting Co., Ltd., in March, 1920, and work was commenced in the following month.

DESIGN.

The new bridge is 720 ft. long between the abutments, and is divided into 18 spans of 40 ft. each. It has a width of 36 ft. between the parapets, providing two footways 5 ft. wide and a carriage-way of 26 ft. between the kerbs. The piers dividing the spans rest on reinforced concrete piles 15 in. square, driven into the bed of the river to a firm set. This has been obtained at varying depths, ranging from 7·62 ft. to 40·17 ft. below ordnance datum, which is about 3 ft. above the bed of the river. Generally, a firm set was obtained between 25 ft. and 35 ft. below O.D. There are four piles to each pier, the two outer piles and the two central piles being fixed in line. The two central piles are 3 ft. centre to centre, and the outer piles 14 ft. from the nearest central pile. The heads of the outer piles are encased in reinforced concrete cylinders 3 ft. in diameter, and the two central piles are enclosed in one concrete casing with cylindrical ends. These casings are sunk into the river bed to varying depths, approximately 3 ft., and are filled in solid around the head of the pile with concrete, all soft material being previously removed from the interior. These form the bases of the four cylindrical columns, each 1 ft. 9 in. in diameter, which carry the main beams. The columns are tied together and cross-braced with reinforced concrete tie-beams 10 in. wide by 14 in. deep. The columns are built *in situ*, and have vertical reinforcing bars attached to and in continuation of the vertical bars in the piles. A few of the tie-beams and braces were pre-cast and built into position when the columns were being erected, but this method was abandoned, it being found better to build them *in situ*. At a height of 16 ft. above the base of the columns, which is approximately high-water mark of ordinary spring tides, a capping is



[*Mr. George A. Phillips, M.Inst.C.E., County Engineer.*

LOUGHOR BRIDGE, SWANSEA. (*See p.* 247.)

formed on the outer columns, and from this level the arched soffits of the main beams spring.

The main longitudinal beams are 12 in. thick by 2 ft. 8 in. deep in the centre, and at the piers the depth is increased to 6 ft. The secondary or cross beams are 18 in. deep by 7 in. wide. There are four main or longitudinal beams and twelve secondary or cross beams to each span. At the piers the cross beam is thickened to 10 in., as it also forms a tie to the head of the columns. Upon these the reinforced concrete decking, 5½ in. thick, forms the floor of the bridge. The decking is extended beyond the outer main beam to form about half the width of the footway. This is supported on a series of shaped brackets, which are really extensions of the secondary or cross beams. Along the outer edge of this decking the moulded base or plinth of the parapet is formed, broken at each pier by a pillar, into which is housed the moulded hand-rail forming the top of the parapet, the space between the hand-rail and plinth being filled in with pierced concrete panels, which are pre-cast.

At the ends of the bridge larger and more important pillars are built to terminate the parapet, and in the centre of the bridge special pillars are built to receive tablets recording the building and formal opening of the bridge.

The abutments are formed by four piles similar to those in the piers, but no columns are built, the main beams landing direct on to the piles. The spaces between the piles are filled in by a reinforced concrete wall supporting the ground. These are built well on the land side behind the stone abutments of the old bridge, which will be left to help protect the new abutments.

Under each footway at the back of the kerb a space is left 18 in. wide by 9 in. deep between the main decking and the underside of the footpath flagging, which forms a duct along the whole length of the bridge on both sides, intended for the accommodation of electric cables and similar services. Provision has been made for carrying electric wires through tubes leading from these ducts up the centre of each pillar of the parapet, so that the bridge may be conveniently wired and electrically lighted at some future time. The footway will be finished with a granolithic surface.

The top of the decking under the carriage-way is to be well painted with hot distilled tar, upon which will be spread a layer of sand 2 in. thick, forming a cushion underneath the tarred macadam roadway, which will be finished with 4 in. of "Tarmac."

The concrete for the piles consists of the following proportions:—7 cwt. of cement, 13½ cu. ft. of sand, and 27 cu. ft. of coarse material; and for the columns, beams, floors, etc., 6 cwt. of cement, 13½ cu. ft. of sand, and 27 cu. ft. of coarse material, where the voids in the aggregate are not greater than 50 per cent. Should the voids be greater, the sand and cement are increased in proportion. The coarse material is specified to be broken or crushed granite or other approved close-grained hard stone, free from dust, etc., and to range in size from ¾ in. to ⅛ in. The cement is British Portland cement of approved manufacture, conforming to British Standard Specification No. 12.

The steel reinforcement consists of plain round steel bars, conforming to the British Standard Specification No. 15. The bars are of varying sizes, from 3/16 in. to 1⅝ in. in diameter, arranged in the ordinary manner, and kept in position by stirrups, etc., formed of smaller size bars bent to the necessary shape.

CONSTRUCTION.

During the building of the bridge it was necessary to provide for the passage of traffic with the least possible interruption. This point was borne in mind in designing and setting out the structure. The arrangement of the piles and columns for the piers enabled the bridge to be built in half-widths. As the new structure is approximately double the width of the old bridge, it was possible to arrange for the additional width on one side of the old structure, and the half-width of the new bridge was capable of being built before any serious disturbance was necessary to the old structure. The southern half was, therefore, built first, and the traffic turned on to it early in March, 1922. The northern half was then proceeded with, the old structure being made use of as far as possible in connection with the temporary staging and demolished as the new structure progressed.

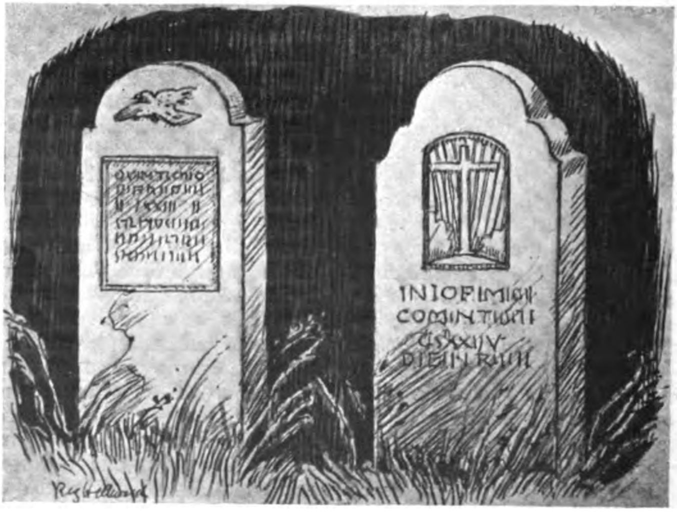
As before stated, the work was actually

commenced in April, 1920, and the making and the driving of the piles proceeded with. No serious difficulty was met with in driving any of the piles, but care had to be exercised in keeping them strictly in line. Only in one instance was a pile diverted seriously out of line while being driven, and in this case an extra pile had to be driven alongside and the two joined at the head within the casing forming the base of the columns, so that the column might be built centrally on the support. At the commencement the piles were driven from one side of the river only, but as time went on the work was carried out from both shores. The cylindrical columns and open construction of the piers present the minimum obstruction to the flow of the tide. As the piles were driven into place the work of casing their heads and forming the bases of the columns followed, and the building of the piers and the erection of the superstructure proceeded, so that the work went on continuously.

In building the parapets the reinforcement of the hand-rail and panels has not been connected to the pillars at the piers, but housed into recesses in the pillars. This has been done so as to permit of free movement in expansion and contraction, particularly as the parapet is the portion of the structure most exposed to the sun. No other provision for expansion has been made, and, judging from the twelve months' experience on the southern half of the bridge, where the sun would have the greatest effect, none appears to be necessary, as no cracks or crevices have been discovered.

The bridge has been designed to carry a normal working super load of 120 lb. per sq. ft., and a rolling load of 48 tons, such as two traction engines, 24 tons each, passing over the bridge as closely as possible at a speed of five miles an hour. The maximum static test load is to be 50 per cent. in excess of the normal super working load of 120 lb. per sq. ft., and the deflection on the tests must not exceed 1-600th part of the span for any part of the construction. These tests have not yet been carried out.

The contract sum for the bridge amounts to £56,665, but this is subject to alteration under a fluctuating clause operating with the rise or fall in the cost of labour and materials.



[Designs by Mr. Reginald Hallward.

HEADSTONES IN CONCRETE.

CEMENT NOTES.

By Our Special Contributor.

Modern Quality Standards of Portland Cement.

It is doubtful whether the degree of precision reached during the past few years in the production of Portland cement has been realised outside the narrow circle of cement manufacturers. Although it must be admitted that the manufacture still has an empirical basis, and there is still some doubt as to the exact chemical compounds present in cement, yet the conditions required to produce a good quality cement are so exactly known that it has become merely a matter of possessing the necessary appliances for a manufacturer to be able to reach and maintain a high standard of quality.

So far as freedom from impurity is concerned, the only item in a chemical analysis of Portland cement that can be classed as impurity is " insoluble matter," and in the product of a modern factory this is less than one-half per cent. A purity exceeding 99½ per cent. is a standard hardly to be improved upon, and one comparing favourably with the products of other industries.

Chemical purity alone, however, does not determine the quality of cement, and another important feature is regularity of composition. The preparation of the mixture of raw materials (by the wet method) has been improved to such an extent that the proportion of carbonate of lime in the " slurry " does not vary by more than 0·2 per cent. during a day. When dealing with raw materials dug from the earth's crust at the rate of a few hundred tons per day this may seem a wonderful achievement, but there is no wizardry about it. The huge mixing basins that are prominent in cement factories usually contain enough mixed raw material to last for several days' manufacture, thus giving the chemists the same period of time for adjusting the mixture to the required composition before it is passed on to the next stage. Time and common sense are all that is required to produce the regularity referred to.

The next stage in the manufacture is the burning or calcination, and so long as the material is heated to a given temperature for a given time the result will be satisfactory. With a continuous kiln, such as a rotary kiln, all that is necessary to produce this result is to feed a constant quantity of raw material mixture (slurry) into the kiln at one end and at the other end to supply heat in the form of a constant stream of coal (ground to a constant fineness) with the necessary volume of hot air for combustion. Instruments for recording temperatures, volumes of air passing, composition of combustion gases, etc., are provided, so that any change in conditions can be immediately detected and adjusted. The result is seen in a constant stream of well-burnt " clinker " issuing from the kiln, and runs of a week or more without a minute's stoppage are achieved.

The final stage in the manufacture is " clinker " grinding, and with modern grinding plant this is performed by passing the requisite quantity of clinker per hour into a mill. The issue of a constant stream of cement of consistent fineness is then practically assured. The fineness is, however, checked by frequent sieve tests.

There is thus no reason why cement made with modern appliances should vary in quality, and, as stated at the outset of this note, the degree of consistency of quality in a Portland cement factory of to-day is remarkable. From a chemical standpoint, cement manufacture is simple, comprising no more than the grinding and mixing of two materials, heating the mixture to a given temperature, and grinding the product. Needless to add, perfection of mixing, burning, and grinding is only attained by the use of extensive plant involving a large consumption of power and heavy charges for renewals which inevitably tend to increase the cost of production in an almost direct ratio with each improvement in quality.

Economy in Cement Manufacture.

Some information upon the work being done by the British Portland Cement Research Association is given in a pamphlet entitled "The Viscosity of Raw Material Slurry used in the Manufacture of Portland Cement" recently issued by the Association. The pamphlet deals with methods of reducing the proportion of water in "slurry" as known by cement manufacturers.

In considering the relative advantages of the wet and dry processes of cement manufacture, the most serious objection to the wet process is the expenditure of fuel that is necessary in the kiln to dry out the 40 per cent. or so of water that is present in the slurry. At least 7 tons of fuel per 100 tons of cement are consumed in this way, and, although in the majority of cases this expense is counterbalanced by the advantages of the wet process over the dry process, it is regarded by manufacturers as one of the hindrances to the cheap production of cement. Hence, in investigating the possibility of reducing the proportion of water in slurry, the British Portland Cement Research Association is performing a distinct service to the industry and indirectly to the cement consumer.

The fact that minute additions of sodium carbonate (washing soda or soda ash) or other salts to clay slurries render the latter more fluid has been known in the pottery trade for many years, but the possibility of utilising the phenomenon in the cement industry does not appear to have been recognised until 1916. Several manufacturers then experimented and found that by adding a few pounds of soda per ton of raw material they were able in some cases to produce a slurry of the usual consistency with from 2 to 6 per cent. less water. With this reduced proportion of water it was possible to reduce the fuel consumption in the rotary kiln. The commercial advantage of the soda addition depends upon the actual reduction in the proportion of water in the slurry and the price of coal; the Research Association pamphlet shows that when 0·075 per cent. of soda ash, costing £10 per ton, is added to slurry and the proportion of water is reduced by 4 per cent. there would be a saving of 15*s.* 10*d.* per 100 tons of clinker if ground coal cost 30*s.* per ton.

The varying effects of the addition of soda to different slurries were investigated, and one of the causes was found to be the soluble salts contained in the raw materials or in the water of the slurry, which reacted chemically with the soda and destroyed its action. There are other causes operating which are at present obscure.

With the information provided by the Research Association before them, cement manufacturers are able to determine very readily for themselves what advantage can be gained in each particular case from adding soda to slurry.

PROPORTIONING CONCRETE MIXTURES.*

THE results of some 50,000 tests carried out by the Structural Materials Research Laboratory, at the Lewis Institute, Chicago, under the direction of Professor Duff A. Abrams, to ascertain the most economical concrete mix to produce a given strength, are set out in a Bulletin published by the Institute, from which we take the following :—

The studies have covered an investigation of the inter-relation of the following factors :—

(1) Consistency (quantity of mixing water) ; (2) Size and grading of aggregates ; and (3) Mix (proportion of cement). The tests have been largely confined to compression tests of concrete and mortars, and have given a new insight into the factors which underlie the correct proportioning of concrete mixtures and show the limitations of older methods. Certain phases of the investigations are still under way.

The following may be mentioned as among the most important principles which have been established with reference to the design of concrete mixtures.

(1) With given concrete materials and conditions of test the quantity of mixing water used determines the strength of the concrete so long as the mix is of a workable plasticity.

(2) The sieve analysis furnishes the only correct basis for proportioning aggregates in concrete mixtures.

(3) A simple method of measuring the effective size and grading of an aggregate has been developed. This gives rise to a function known as the "fineness modulus" of the aggregate.

(4) The fineness modulus of the aggregate furnishes a rational method for combining materials of different size for concrete mixtures.

(5) The sieve analysis curve of the aggregate may be widely different in form without exerting any influence on the concrete strength.

(6) Aggregate of equivalent concrete-making qualities may be produced by an infinite number of different gradings of a given material.

(7) Aggregates of equivalent concrete-making qualities may be produced from materials of widely different size and grading.

(8) In general, fine and coarse aggregates of widely different size or grading can be combined in such a manner as to produce similar results in concrete.

(9) The aggregate grading which produces the strongest concrete is not that giving the maximum density (lowest voids). A grading coarser than that giving maximum density is necessary for highest concrete strength.

(10) The richer the mix, the coarser the grading should be for an aggregate of given maximum size ; hence, the greater the discrepancy between maximum density and best grading.

(11) A complete analysis has been made of the water-requirements of concrete mixes. The quantity of water required is governed by the following factors :— (*a*) The condition of "workability" of concrete which must be used—the relative plasticity or consistency ; (*b*) the normal consistency of the cement ; (*c*) the size and grading of the aggregate—measured by the fineness modulus ; (*d*) the relative volumes of cement and aggregate—the mix ; (*e*) the absorption of the aggregate ; (*f*) the contained water in aggregate.

(12) There is an intimate relation between the grading of the aggregate and the quantity of water required to produce a workable concrete.

(13) The water content of a concrete mix is best considered in terms of the volume of the cement—the water-ratio.

(14) The shape of the particle and the quality of the aggregate have less influence on the concrete strength than has been reported by other experimenters.

EFFECT OF QUANTITY OF MIXING WATER ON THE STRENGTH OF CONCRETE.

Fig. 1 shows the relation between the compressive strength and the water content for 28-day tests of 6 by 12-in. concrete cylinders. Mixes from 1 : 15 to neat cement were used ; each mix was made up of aggregates ranging in size from 14-mesh sand up to 1½-in. gravel ; a wide range in consistencies was used for all mixes and gradings. The water content of the concrete is expressed as a ratio to the volume of cement, considering that the cement weighs 94 lb. per cu. ft. Distinguishing marks are used for each mix, but no distinction is made between aggregates of different size or different consistencies.

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* In our issues for October and November last we gave a series of tables, prepared by Professor Duff Abrams and Mr. Stanton Walker, giving the proportions necessary to produce concrete of different compressive strengths from 1,500 lb. per sq. in. to 4,000 lb. per sq. in.

When the compressive strength is plotted against the water ratio in this way a smooth curve is obtained, due to the overlapping of the points for different mixes. Values from dry concretes have been omitted. If these were used we should obtain a series of curves dropping downward and to the left from the curve shown. It is seen at once that the size and grading of the aggregate and the quantity of cement are no longer of any importance except in so far as these factors influence the quantity of water required to produce a workable mix. This gives us an entirely new conception of the function of the constituent materials entering into a concrete mix, and is the most basic principle which has been brought out in our studies of concrete.

The equation of the curve is of the form,

$$S = \frac{A}{B^x} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (1)$$

where S is the compressive strength of concrete and x is the ratio of the volume of water to the volume of cement in the batch. A and B are constants whose values depend on the quality of the cement used, the age of the concrete, curing conditions, etc. This equation expresses the law of strength of concrete so far as the proportions of materials are concerned. It is seen that for given concrete materials the strength depends on only one factor—the ratio of water to cement. Equations which have been proposed in the past for this purpose contain terms which take into account such factors as quantity of cement, proportions of fine and coarse aggregate, voids in aggregate, etc., but they have uniformly omitted the only term which is of any importance; that is, the water.

For the conditions of these tests, equation (1) becomes,

$$S = \frac{14{,}000}{7^x} \quad . \quad . \quad . \quad . \quad . \qquad (2)$$

The relation given above holds so long as the concrete is not too dry for maximum strength and the aggregate not too coarse for a given quantity of cement; in other words, so long as we have a workable mix. Other tests made in this laboratory have shown that the character of the aggregate makes little difference so long as it is clean and not structurally deficient. The absorption of the aggregate must be taken into account if comparison is being made of different aggregates.

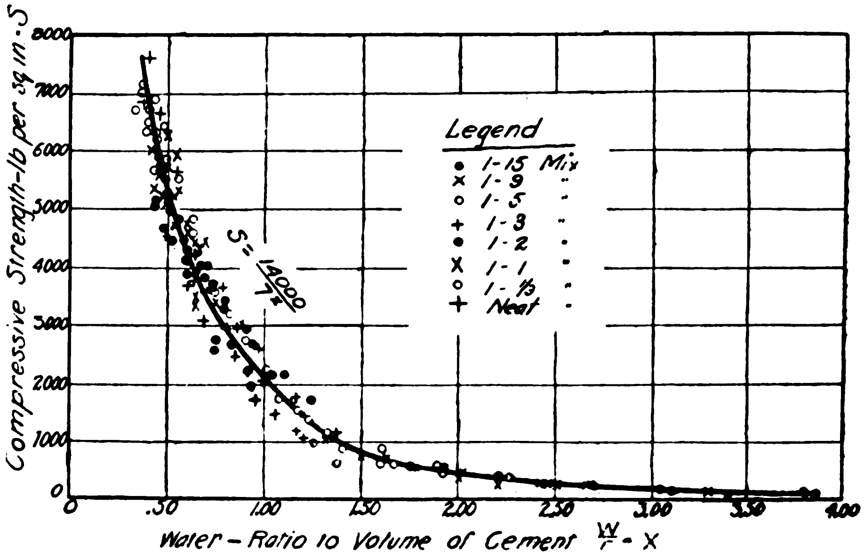


FIG. 1. RELATION BETWEEN STRENGTH OF CONCRETE AND WATER CONTENT; 28-DAY COMPRESSION TESTS OF 6 IN. BY 12 IN. CYLINDERS (SERIES 83).

TABLE 1.—METHOD OF CALCULATING FINENESS MODULUS OF AGGREGATES.

Each sieve has a clear opening double that of the preceding sieve. The sieve analysis may be expressed in terms of volume or weight. The fineness modulus of an aggregate is the sum of the percentages given by the sieve analysis divided by 100.

			Sieve Analysis of Aggregates Per Cent of Sample Coarser than a Given Sieve.						
			Sand			Pebbles			Concrete
Sieve Size	Size of Square Opening in.	Size of Square Opening mm.	Fine (A)	Medium (B)	Coarse (C)	Fine (D)	Medium (E)	Coarse (F)	Aggregate (G)*
100-mesh	.0058	.147	82	91	97	100	100	100	98
48-mesh	.0116	.295	52	70	81	100	100	100	92
28-mesh	.0232	.59	20	46	63	100	100	100	86
14-mesh	.046	1.17	0	24	44	100	100	100	81
8-mesh	.093	2.36	0	10	25	100	100	100	73
4-mesh	.185	4.70	0	0	0	86	95	100	71
⅜-in.	.37	9.4	0	0	0	51	66	86	49
¾-in.	.75	18.8	0	0	0	9	25	50	19
1½-in.	1.5	38.1	0	0	0	0	0	0	0
Fineness Modulus............			1.54	2.41	3.10	6.46	6.86	7.36	5.74

* Concrete aggregate "G" is made up of 25 per cent. of sand "B" mixed with 75% of pebbles "E." Equivalent gradings would be secured by mixing 33% sand "B" with 67% coarse pebbles "F"; 28% "A" with 72% "F," etc. The proportion coarser than a given sieve is made up by the addition of these percentages of the corresponding size of the constituent materials.

The strength of the concrete responds to changes in water, regardless of the reason for these changes. The water-ratio may be changed due to any of the following causes: (1) Change in mix (cement content); (2) change in size or grading of aggregate; (3) change in relative consistency; (4) any combination of (1) to (3).

In certain instances a 1 : 9 mix is as strong as a 1 : 2 mix, depending only on water content. It should not be concluded that these tests indicate that lean mixes can be substituted for richer ones without limit. We are always limited by the necessity of using sufficient water to secure a workable mix. So in the case of the grading of aggregates. The workability of the mix will in all cases dictate the minimum quantity of water that can be used. The importance of the workability factor in concrete is therefore brought out in its true relation.

The problem of designing concrete mixes resolves itself into this: To produce a workable concrete which has a given water-ratio using a minimum quantity of cement; or the converse, to produce a workable concrete with a minimum water-ratio using a given quantity of cement. The methods for securing the best grading of aggregate and the use of the driest concrete which is workable are thus seen to be only devices which enable us to accomplish the above-mentioned results.

FINENESS MODULUS OF AGGREGATE.

The experimental work carried out in the Laboratory has given rise to what we term the fineness modulus of the aggregate. This function furnishes a method of measuring the size and grading of the aggregate. It may be defined as the sum of the percentages in the sieve analysis of the aggregate divided by 100. The sieve analysis is determined by using the following sieves from the Tyler standard series: 100, 48, 28, 14, 8, 4, ⅜-in., ¾-in., and 1½-in. These sieves are made of square-mesh wire cloth. Each sieve has a clear opening just double the width of the preceding one. The exact dimensions of the sieves and the method of determining the fineness modulus will be found in Table 1. It will be noted that the sieve analysis is expressed in terms of the percentages of material by volume of weight coarser than each sieve.

A well-graded torpedo sand up to No. 4 sieve will give a fineness modulus of about 3·00; a coarse aggregate graded 4-1½ in. will give fineness modulus of about 7·00; a mixture of the above materials in proper proportions for a 1 : 4 mix will have a fineness modulus of about 5·80. A fine sand such as drift-

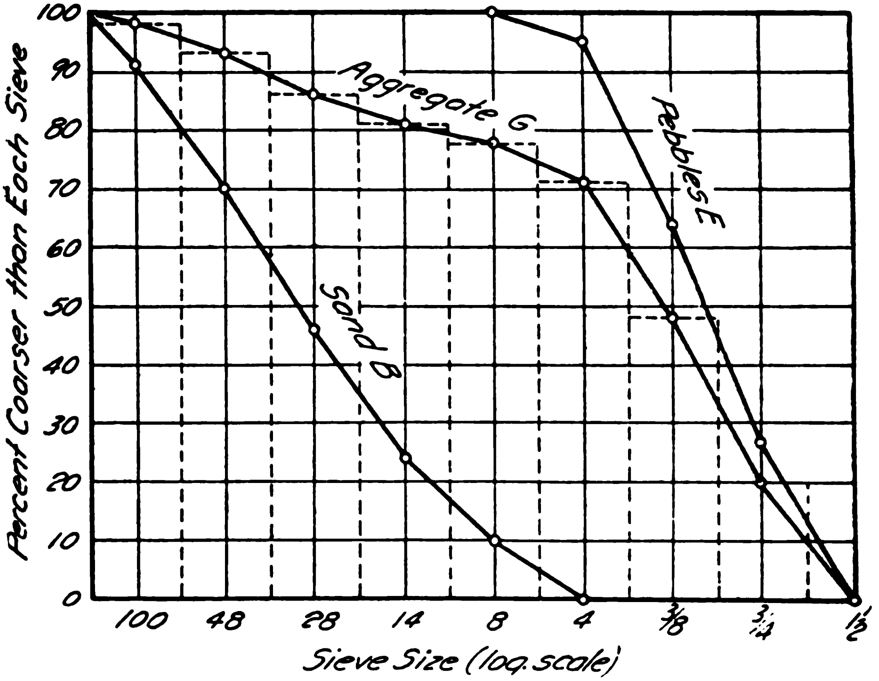


FIG. 2. METHOD OF PLOTTING SIEVE ANALYSIS OF AGGREGATES (SIEVE ANALYSIS CURVES FOR AGGREGATES B, E, AND G IN TABLE 1).

sand may have a fineness modulus as low as 1·50.

SIEVE ANALYSIS OF AGGREGATES.

There is an intimate relation between the sieve analysis curve for the aggregate and the fineness modulus; in fact, the fineness modulus enables us for the first time properly to interpret the sieve analysis of an aggregate. If the sieve analysis of an aggregate is plotted in the manner indicated in *Fig.* 2; that is, using the per cent. coarser than a given sieve as ordinate, and the sieve size (plotted to logarithmic scale) as abscissa, the fineness modulus of the aggregate is measured by the area below the sieve analysis curve. The dotted rectangles for aggregate "G" show how this result is secured. Each elemental rectangle is the fineness modulus of the material of that particular size. The fineness modulus of the graded aggregate is then the summation of these elemental areas. Any other sieve analysis curve which will give the same total area corresponds to the same fineness modulus, and will require the same quantity of water to produce a mix of the same plasticity and gives concrete of the same strength so long as it is not too coarse for the quantity of cement used. The fineness modulus may be considered as an abstract number; it is in fact a summation of volumes of material. There are several different methods of computing it, all of which will give the same result. The method given in Table 1 is probably the simplest and most direct.

Some of the mathematical relations involved are of interest. The following expression shows the relation between the fineness modulus and the size of the particle:—

$$m = 7{\cdot}94 + 3{\cdot}32 \log. d \quad .\ .\ .\ . \quad (3)$$

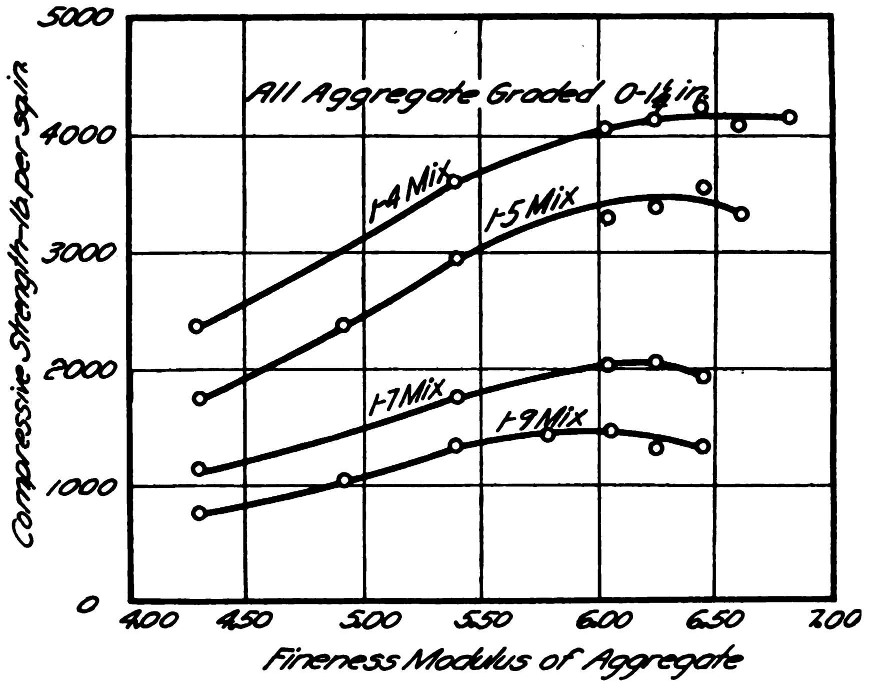
Where m = fineness modulus

d = diameter of particle in inches.

This relation is perfectly general so long as we use the standard set of sieves mentioned above. The constants are fixed by the particular sizes of sieves used and the units of measure. Logarithms are to the base 10. This relation applies to a single-size material or to a given particle. The fineness modulus is then a logarithmic function of the diameter of the particle. This formula need not be used with a graded material, since the value can be secured more easily and directly by the method used in Table 1. It is applicable to graded materials provided the relative quantities of each size are considered, and the diameter of each group is used. By applying the formula to a graded material we would be calculating the values of the separate elemental rectangles shown in *Fig.* 2.

FINENESS MODULUS STRENGTH RELATION FOR CONCRETE.

Many different series of tests have shown that for a given plastic condition of concrete and the same mix there is an intimate relation between the fineness modulus of the aggregate and the strength and other properties of the concrete. We have seen that the reason for this result is found in the fact that the fineness modulus simply reflects the changes in



Sand and pebble aggregate graded 0–1½ in.; 28-day compression tests of 6-in. by 12-in. cylinders (Series 78).

The sieve analyses of aggregates are given below:—

Range in Size	Fineness Modulus	Per Cent Coarser than Each Sieve									
		100	48	28	14	8	4	⅜	¾	1½	2
0-1¼	4.30	89	82	72	62	51	38	25	11	0	..
"	4.93	95	89	82	73	61	47	32	14	0	..
"	5.40	98	94	88	80	69	55	38	18	0	..
"	6.04	99	98	95	90	81	68	49	24	0	..
"	6.25	100	99	97	92	85	72	53	27	0	..
"	6.45	100	99	98	95	88	77	58	30	0	..
"	6.60	100	100	99	96	91	80	62	32	0	..
"	6.82	100	100	99	98	94	86	68	37	0	..

FIG. 3. RELATION BETWEEN FINENESS MODULUS OF AGGREGATE AND STRENGTH OF CONCRETE.

water-ratio necessary to produce a given plastic condition.

Figs. 3 and 4 give the results of compression tests which bring out the relation between the strength of the concrete and the fineness modulus of the aggregate. It will be noted from *Fig.* 3 that a separate curve may be drawn for each mix. In each case there is a steady increase in the compressive strength of the concrete as the fineness modulus of the aggregate increases, until a certain value is reached which corresponds to a maximum point. It will be noted also that this maximum point corresponds to higher and higher values of fineness modulus as the quantity of cement in the mix is increased. In other words, the maximum strength comes at a fineness modulus of about 5·80 for the 1 : 9 mix and about 6·40 for the 1 : 4 mix. In these tests the different values of the fineness modulus were secured by using a preponderance of the coarser sizes, but in all cases maintaining the same limiting size, that is, 1¼ in.

In *Fig.* 4 is a similar relation between the strength and the fineness modulus, except that no maximum point is found. This condition arises from the fact that the maximum size of the aggregate is

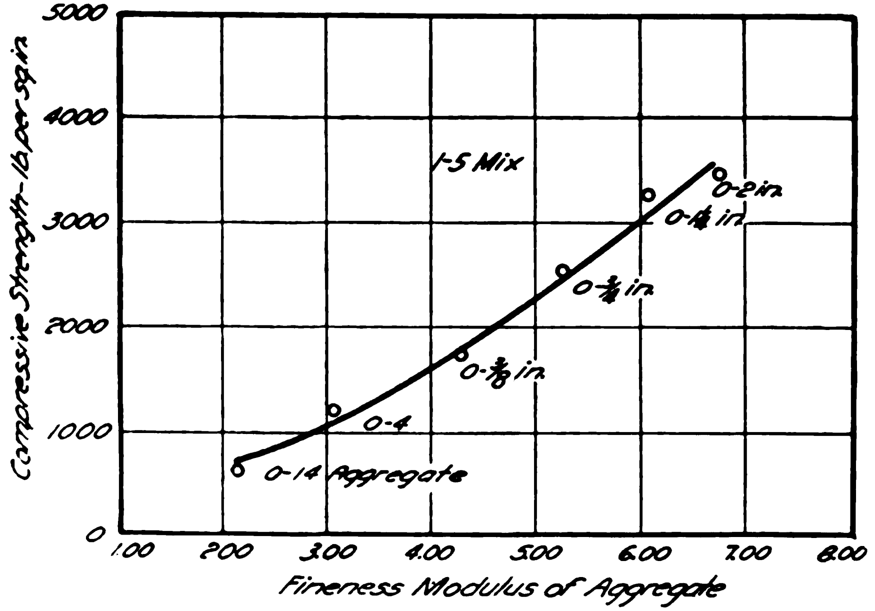


FIG. 4. RELATION BETWEEN FINENESS MODULUS OF AGGREGATE AND STRENGTH OF CONCRETE.

[28-day compression tests of 6-in. by 12-in. cylinders (Series 78). Sand and pebble aggregate graded to sizes shown. The contrast between the relation shown by these tests and those in Fig. 3 should be noted.]

The sieve analyses of aggregates are given below :—

Range in Size	Fineness Modulus	Per Cent Coarser than Each Sieve									
		100	48	28	14	8	4	⅜	¾	1½	2
0-14	2.16	95	84	37	0	..	..	..	..	..	..
0- 4	3.06	96	90	62	40	18	0	..	..	..	..
0- ⅜..............	4.26	96	91	83	71	54	31	0	..	..	..
0- ¾..............	5.24	98	96	91	83	71	54	31	0	..	..
0- 1¼..............	6.04	99	98	95	90	81	68	49	24	0	..
0- 2	6.72	100	99	97	94	87	77	62	42	14	0

FIG. 4. RELATION BETWEEN FINENESS MODULUS OF AGGREGATE AND STRENGTH OF CONCRETE.

increasing without changing the type of the sieve analysis curve, consequently the fineness modulus strength curve continues to rise indefinitely. The height to which the curve rises is limited only by the maximum size of aggregate which may be used. It is important to note that there is no conflict between the indications of *Figs.* 3 and 4.

For all practical purposes and for ordinary ranges in concrete mixes the fineness modulus strength relation may be assumed as a linear one. The comparison of the true and approximate

TABLE 2.—EFFECT OF GRADING AGGREGATES ON THE STRENGTH OF CONCRETES.

Compression tests of 6-in. by 12-in. concrete cylinders.

Mix, 1 : 5 by volume; age at test, 28 days; stored in damp sand; tested damp.

Aggregates, sand and pebbles screened to different sizes and recombined to conform to predetermined sieve analysis. The aggregates were made up so as to give the widest variations in the grading of the particles. All gradings had one common property in that the fineness modulus was exactly the same, viz., $m = 6{\cdot}04$. The fineness modulus of the aggregate is the sum of the percentages in the sieve analysis divided by 100.

The same quantity of water was used in all specimens of a given consistency. The 110% consistency contains 10% more water than the 100%.

Each specimen was made from a different batch.

Each value in the strength tests is the average from five tests made on different days (from Series 78).

Ref. No.	Sieve Analysis of Aggregate Per Cent Coarser than Each Sieve											Fineness Modulus of Aggregate	Surface Area of Aggregate sq. in.		Compressive Strength of Concrete at 28 days (lb. per sq. in.)	
	100	48	28	14	8	4	3/8	3/4	1 1/4	1 1/2	2		per lb. of Aggregate	per g. of Cement	100% Consistency	110% Consistency
40	99	98	95	90	81	68	49	24	0			6.04	602	8.8	3,300	2,890
259	99	98	96	92	84	67	46	22	0			6.04	569	8.2	2,950	2,650
260	98	97	93	88	80	67	52	29	0			6.04	764	11.4	3,120	2,760
261	97	94	91	85	77	67	58	35	0			6.04	999	15.2	3,140	2,790
262	95	92	87	82	75	67	67	39	0			6.04	1,292	20.1	3,100	2,800
263	95	90	84	78	73	67	62	55	0			6.04	1,451	23.0	2,830	2,740
264	95	89	82	75	67	67	67	62	0			6.04	1,565	25.2	2,680	2,580
265	100	97	91	79	72	67	58	40	0			6.04	761	11.9	3,070	2,690
266	100	97	93	88	83	67	50	27	7	0		6.04	616	9.0	3,080	2,790
267	99	97	94	86	77	67	47	27	16		0	6.04	709	10.5	3,150	2,710
268	98	95	90	83	83	83	50	22	0			6.04	834	12.6	3,080	2,500
269	98	94	90	86	83	80	55	18	0			6.04	898	13.3	3,050	2,550
270	96	90	80	80	80	80	60	39	0			6.04	1,391	21.5	2,970	2,550
271	100	96	92	87	81	75	50	23	0			6.04	672	10.0	2,930	2,710
272	95	91	87	82	77	73	59	40	0			6.04	1,315	20.2	3,000	2,580
273	99	95	88	80	76	73	61	32	0			6.04	911	13.9	2,950	2,740
274	90	85	81	78	75	73	66	56	0			6.04	1,992	31.3	2,680	2,440
275	100	93	82	73	73	73	63	47	0			6.04	1,076	16.7	2,820	2,620
276	100	100	100	92	81	60	45	26	0			6.04	390	5.6	3,040	2,780
277	100	98	95	90	80	60	50	31	0			6.04	557	8.3	2,900	2,770
278	100	99	96	92	84	55	50	28	0			6.04	483	7.0	2,940	2,750
279	100	99	96	91	80	50	50	38	0			6.04	514	7.6	3,080	2,750
280	98	84	84	84	84	57	57	57	0			6.04	1,276	19.7	3,000	2,780
281	99	98	91	86	80	76	38	38	0			6.04	701	10.4	2,940	2,700
282	99	98	91	86	80	76	46	30	0			6.04	697	10.2	3,020	2,660
283	99	98	91	86	80	76	61	15	0			6.04	689	10.1	2,930	2,670
284	99	98	91	85	80	76	67	8	0			6.04	685	9.9	2,970	2,630
Average												6.04	904	13.8	2,990	2,690
Minimum Value												....	390	5.6	2,680	2,440
Maximum Value												....	1,992	31.3	3,300	2,890
Mean Variation from Average—per cent												...	34.4	37.2	3.41	3.04

relation is brought out in the discussion of the "Water Formula." A given value for the fineness modulus of an aggregate can be secured with any combination of percentages in the sieve analysis which gives the same total; consequently, an infinite variety of gradings may be found which give aggregate of the same concrete strength. Table 2 gives the results of groups of tests which bring out the wide variation which may be made in the grading of aggregate without producing any essential variation in the concrete strength. Twenty-seven different gradings of the same aggregate were made. These gradings covered the widest possible range, but they had one property in common: that is, a fineness modulus of 6·04. All specimens were mixed with the same quantity of cement and water. Separate sets of specimens were made of two different consistencies.' The mean variation from the average strength is about 3 per cent. Table 2 also furnishes some interesting data on the surface-area method of proportioning aggregates. It is seen that there is the widest variation in the surface area of the aggregate without any appreciable difference in the concrete strength. Our studies have clearly shown that surface area is not a satisfactory basis for proportioning aggregates.

(*To be continued.*)

THE "TUDSBERY" STRESS INDICATOR.

WE have received from the makers (Messrs. C. F. Casella & Co., Ltd., of No. 50 Parliament-street, S.W.) a pamphlet describing a stress indicator, the object of which is to ascertain at a glance actual stresses in bridge members, structures of various kinds and materials of construction. It consists of a dial which will show any variation in length between two points at a given distance, usually 8 ft., the dial being connected with two wooden rods projecting 4 ft. each side, and the ends of the rods being clamped on to the member to be tested; the relative movement of the two rods can be recorded with an accuracy of 1–4000 part of an inch. If the stress is constant within the length measured, it is a simple function of the elongation recorded, and in this case the instrument should be very useful in ascertaining what is actually taking place in the member under consideration under various conditions of loading. The instrument is stated to have been used with success for testing the Queensferry Bridge over the River Dee, and is intended to cover all materials, including reinforced concrete.

In the case of reinforced concrete it should, however, be observed that a certain caution in using the instrument is necessary if useful results are to be obtained, inasmuch as the readings in most cases will give average elongations and average stresses; strictly speaking, it is only in cases of pure tension or compression that the instrument, being an extensometer, gives the stress correctly, as a function of the elongation. In members subject to bending or combined bending and axial stress, the moment will vary along the member, and with it the stress and the elongation in the extreme fibres. The dial will record the average extension, and consequently the average stress, on the length to which it is applied; as, however, this length can be cut down to 2 ft. by varying the wooden rods it should be possible in most cases to obtain a useful average when the variation in moment is not too abrupt. In the case of short-span beams and slabs where the moment varies very quickly, and it would be difficult to obtain readings giving any real indications of the actual stresses, an excellent way of applying the instrument would be as a deflectometer where the extremely sensitive readings would record movements with great accuracy; a simple calculation would enable the stress to be found from the deflection measured. The price of the instrument is £25 10*s*.

SUPPLEMENT

ON THE

ALL-CONCRETE ROAD

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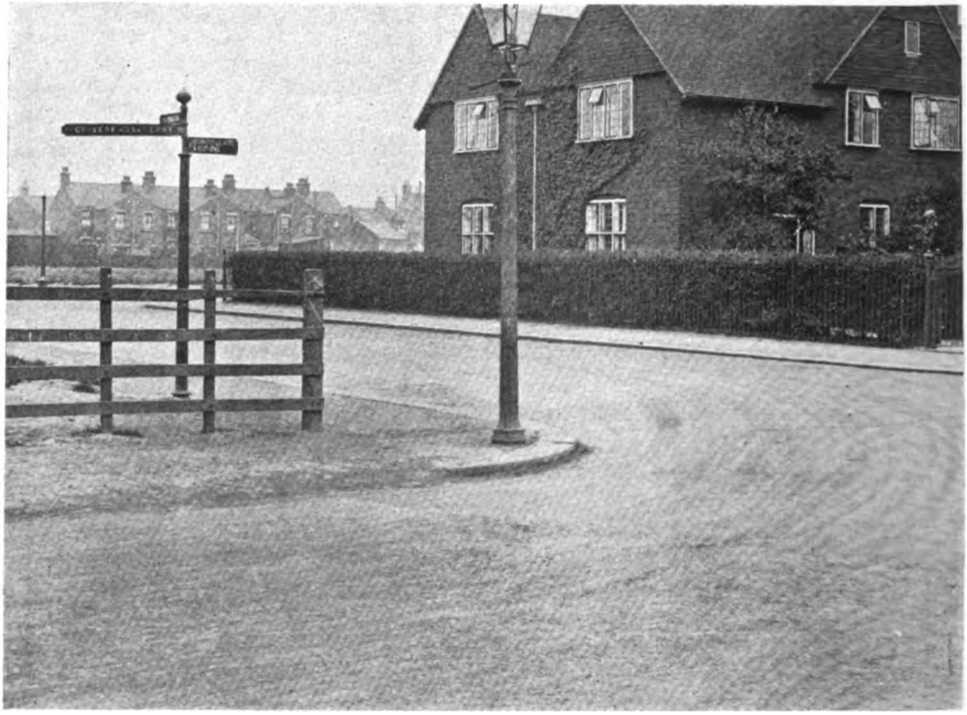
CONDITIONS FOR SUCCESS IN CONCRETE ROAD CONSTRUCTION.

In connection with the use of concrete for road covering there seems to have arisen an atmosphere of suspicion and even mystery, for which it is somewhat difficult to account. The application of concrete to all other types of constructional work is regarded largely as a matter of calculation to decide the required dimensions. But with concrete roads the impression appears to be abroad that there are other rules which govern the subject and that the conditions for success are to some extent unknown and different from those operating in ordinary concrete work. To what extent this impression is well founded may perhaps be determined by considering the conditions a concrete road is required to withstand.

It is desirable that a concrete road surface should have three qualities, namely (1) strength to bear the loads it has to carry without fracture, (2) resistance to the wear arising from wheeled and horse traffic, and (3) freedom from cracking.

With regard to the first-mentioned quality of strength, it is obvious that this is a matter of design which presents no difficulty to the concrete designer. There have indeed been so many concrete road *beds* successfully laid down that



[*Mr. S. A. Hill-Willis, Borough Engineer.*

Reinforced Concrete Road, laid in 1917, for the Tilbury Urban District Council.

[This road is in two courses, with a total thickness of 7 in. The bottom course, of 5½ in., consists of 1 part 1-in. to ¾-in. ballast, 1 part ¾-in. to ⅜-in. ballast, 1 part ¼-in. ballast, 1 part pit sand passing a 1/16th screen, and 1 part Portland cement; the top course, 1½ in. thick, consists of 1 part Guernsey granite chippings, 1 part pit sand and 1 part Portland cement.]

[*Mr. S. A. Hill-Willis, Borough Engineer.*

All-Concrete Road at Tilbury.

the design so far as strength is concerned may almost be said to be stereotyped, although in this case the question of resistance to wear has not arisen. If road surveyors believed that the success of a concrete road depended solely on strength there would probably be very little hesitation in adopting concrete roads.

The next quality is resistance to wear, and this is probably the stumbling-block to those without experience. It may in fact be admitted that this is the most critical condition upon which concrete roads must stand or fall. The hesitation in accepting concrete as a wear-resistant was perhaps natural in the absence of any generally-accepted data upon the subject, and even now, when it cannot be denied that there are many successful concrete roads in existence, the hesitation may still linger because the conditions which lead to success are not clearly defined. In effect, the road engineer requires to know what is necessary to make concrete wear-resisting. For the evidence on this point we have to thank the investigators at the Structural Materials Research Laboratory at Chicago, who set out to determine experimentally the conditions which make concrete wear-resisting. The conclusion to be drawn from their exhaustive investigation can be expressed in the one word "quality." The most wear-resisting concrete is that with the highest quality as expressed in compressive strength, from which it follows that a satisfactory concrete road surface demands a strong concrete.

The conditions that produce a strong and therefore wear-resisting concrete are well known, and may be summarised as a high proportion of cement, no excess of water beyond that required to produce a plastic concrete, thorough mixing, graded clean aggregate, proper curing (i.e., keeping the concrete damp during the period of early hardening), and age. A further assurance of success can be obtained by applying to the surface of the road a non-proprietary surface hardener, such as sodium silicate solution. Such treatment also has the advantage of accelerating the hardening of the concrete and so reducing the period during which traffic must be excluded from the road. The American investigations have thus dispelled the atmosphere of mystery that has surrounded concrete road construction, and have brought this branch of engineering practice within the region of common sense again.

The third quality needed in a concrete road is freedom from cracking, which results from the fact that as concrete must be mixed with more water than the cement can absorb contraction is bound to occur when the excess water evaporates. The tendency to crack that follows on contraction can be mitigated by preventing evaporation (by keeping the concrete damp) for a fortnight or more until the concrete is strong enough to resist the stresses that occur with contraction. Restriction of water for concrete mixing to the minimum required for plasticity is, of course, also of great importance in this connection. Further assistance may be given by reinforcement, such as steel mesh, as near the surface as is compatible with protection from corrosion. In properly-made concrete roads cracks are the exception rather than the rule, and where they do occur they are generally so small that a pen-knife cannot be inserted in them. Such cracks are comparatively harmless if filled up with pitch or similar semi-plastic material and the annual renewal of this filling, at the cost of a few pounds per mile, constitutes the only repair charge of a good concrete road. Under certain conditions, however, it is possible to avoid the irregular cracks by dividing up the road into lengths separated by plastic jointing material, making in effect an artificial transverse crack in the concrete surface at regular intervals (say, 50 ft.), which does away with the suggestion of any unsightliness associated with irregular cracks which might form naturally.

Regarded from a common-sense standpoint, then, the secret of success with concrete roads is to employ strong concrete ; every precaution that can be taken to ensure good concrete will justify itself in the life of the road. Unless a road surveyor can rely upon producing a strong concrete he would be well advised to choose some other material for his road surface. Money spent upon obtaining a strong concrete road surface will avoid the initial and recurring expenditure of applying tar or other bitumastic covering, and will prove an excellent investment.

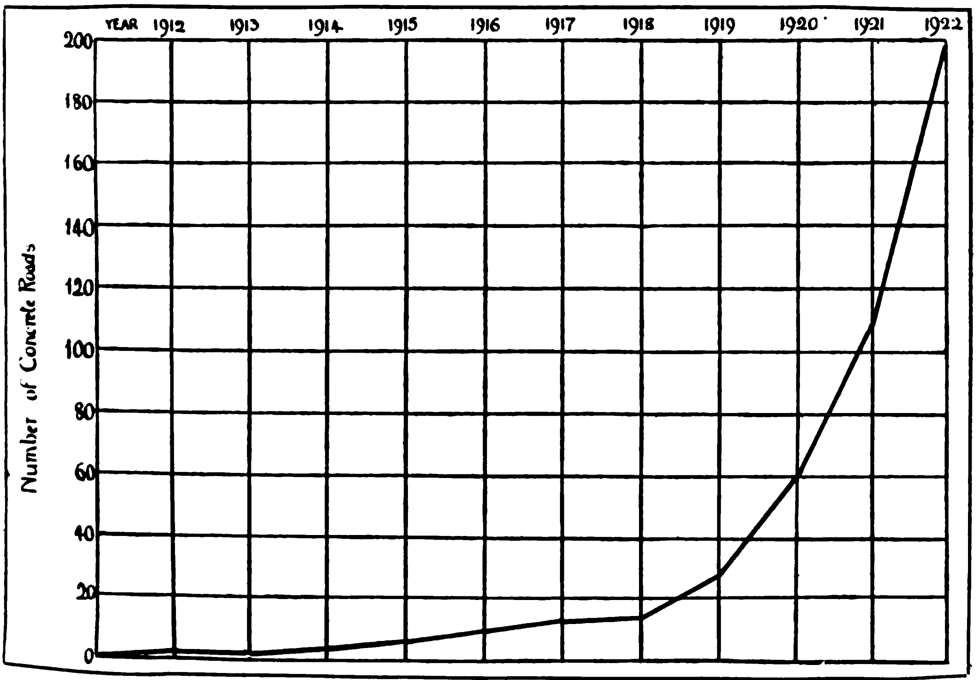
[*Mr. Alfred Dryland, County Engineer of Middlesex :*
Mr. Cuthbert Brown, District Engineer.

REINFORCED CONCRETE ROAD AT EDMONTON.

THE GROWTH OF CONCRETE ROADS IN THE UNITED KINGDOM.

THE remarkable increase in popularity of the all-concrete road in this country is shown by the graph on this page. Previous to 1912 there were in the United Kingdom no concrete roads as we understand them to-day, whereas at the end of 1922 some 200 had been built. Like all other types of roads, the development of the all-concrete road was greatly hampered, if not stopped altogether, by the war conditions between 1914 and 1918. As will be seen from the curve, at the outbreak of war not more than three or four roads of this type had been laid, and at the time of the Armistice that number had only increased to less than twenty. It will be seen, however, that when the restrictions on road construction were removed the number which had been laid during six years was doubled in a few months, and in the four years 1919–22 the number laid during the previous six years had increased by 900 per cent. These figures are a striking indication of the value Borough Surveyors and others now attach to concrete roads, and the following list of roads which have been built during the past ten years, with the date when laid, may be useful to those who may wish to inspect them and to see the effect of traffic after a period of years:

YEAR.	PLACE.
1912	Robert's Lane, Chester.
1913	Nil.
1914	Whitefriars, Chester. Fish Quay, Padstow.
1915	London–Dover Road, Gravesend. Mountnessing, Essex. Glengormly, Belfast.
1916	Marine Drive, Exmouth. Canmore Street, Dunfermline. A road at Tilbury.
1917	Southampton Dock Road. Tilbury Dock Road. Road at Air Station, Kent.
1918	Plymouth Street, Merthyr Tydfil.
1919	6 roads in the Borough of Southwark. Totnes–Paignton. Newport–Cardiff.



THE GROWTH OF CONCRETE ROADS IN THE UNITED KINGDOM FROM 1912 TO 1922.

1919 Brecon–Abergavenny.
Musgrave Channel Road, Belfast.
London Road, Portsmouth.
Bedhampton Waterworks Road, Portsmouth.

1920 17 roads in Southwark.
6 roads in Manchester.
New King Street, Deptford.
Foregate Street, Chester.
Eastgate, Chester.
Buckhurst Hill, Essex.
Longhoughton, Northumberland.
Wideopen, Northumberland.
Morgan Street, New Tredegar.
George Street, Prestwich.
Custom House Dock Road, Dublin.
Main Road, Skegness Housing Estate.
Station Road, Taunton.

1921 23 roads in Southwark.
5 roads in Middlesbrough.
3 roads in Leeds.
2 roads in Manchester.
Lower Bristol Road, Bath.
Esplanade Extension, Southsea.
Rhyl Promenade.
Western Avenue, Acton.
Gelligaer, Cardiff.
Ruthin.
Gilfach Main Road, Bargoed.
Water Street, Aberavon.
Housing Scheme Road, Newark.
Polegate-Willington Road, Eastbourne.
Bridgwater Road, North Petherton.
Bristol-Avonmouth Low Level Road.
Crayford.
Sible Headingham, Essex.
Glenboig Road, Airdrie, Lanarkshire.

1922 20 roads in Southwark.
31 streets in Sheffield.
14 roads in Middlesbrough.
12 roads in Manchester.
Two sections of the North Circular Road, London.

1922 Bury St. Edmunds.
Goole.
Barnstaple.
Estate Road, Hales (2½ miles).
Blaenavon.
Tavistock Road, Plymouth.
6 roads, Ipswich Housing Estate.

1923 2 roads in Middlesbrough.
5 roads in Gosport.
Douglas (Isle of Man).
Huddersfield Road, Bradford.
Coastal Road, Flintshire.
Marina Promenade, Ramsgate.

CONCRETE ROADS UNDER CONSTRUCTION.

Southwood Road, Abersychan.
Five roads in Southwark.
Gronant-Prestatyn (Flintshire).
Cambridge Road (Tottenham to Waltham Cross).
St. Barnabas Road, Middlesbrough.

CONCRETE ROADS IN CONTEMPLATION.

Northfleet, near Rosherville.
Grimsby Road, Cleethorpes.
Further roads at Tilbury.
East to West Road, Surrey.

Summary.

Year.	Number laid.	Total up to end of year.
1912	1	1
1913	0	1
1914	2	3
1915	3	6
1916	3	9
1917	3	12
1918	1	13
1919	13	26
1920	34	60
1921	48	108
1922	90	198

In addition to the above, which are all public roads, 140 concrete roads have been laid in industrial works and military camps.

OPINIONS ON THE ALL-CONCRETE ROAD.

As with all comparatively new methods or materials, there is a natural hesitancy to adopt all-concrete roads until they have been fully tried out. The following expressions of opinion by Borough Surveyors and others who have laid all-concrete roads, should, we think, be valuable as showing the opinions of those who have had experience of the latest type of road construction:—

ANTRIM.

After it had been in use for five years, Mr. D. Megard, County Surveyor of Antrim, stated that a short section of all-concrete road at Belfast (Antrim) was in very fair order, and there had been little expenditure on it since it was laid except the cost of tar-spraying. The road is subject to heavy traffic.

CHESTER.

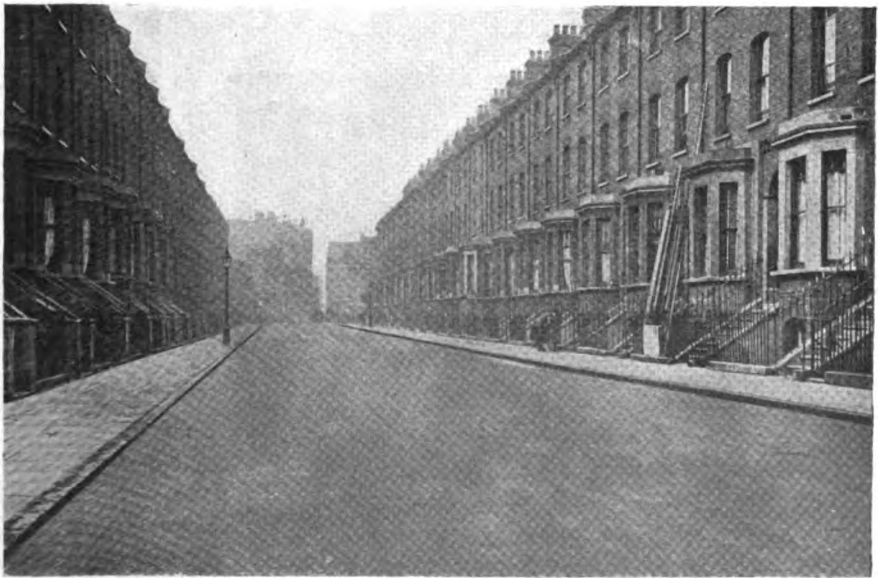
An all-concrete road, 950 ft. long and carrying in 1913 about 60 tons of traffic per day, was laid in Chester in 1912 under the supervision of Mr. Matthews Jones, late City Surveyor. When this road was inspected in 1914 the surface was in "perfect condition"; it was tar-sprayed in 1915, and when it was inspected again in 1919 the surface was stated to be "in complete order, and there was not the slightest sign of the dressing lifting in any way." A further stretch of all-concrete road having been laid in Chester in 1914 the City Surveyor stated that the Highways Committee was so satisfied with the results obtained that it decided to lay a reinforced all-concrete road right through the main thoroughfare, which in 1919 carried 1,500 tons of traffic per day. Part of this work has now been done.

CRAYFORD.

Mr. Walter F. Bickford, Engineer and Surveyor of the Crayford Urban District Council, writes:—

"The length of the first section of reinforced concrete road carried out was 300 ft., and the width 21 ft. between kerbs, and was laid in alternate bays in one course. The thickness of concrete was 7 in., reinforced by Walker-Weston double-layer 4-in. reinforcement, the top wire 2 in. from the surface. The aggregate was Thames ballast in the proportion of 6 to 1. The work was done entirely by hand labour. The surface was composed of 2-in. granite chippings

(*Continued on page* 270.)



[*Mr. Arthur Harrison, Borough Engineer and Surveyor.*

REINFORCED CONCRETE ROAD AT SOUTHWARK. (*See p.* 273.)

THE MAKI ALL-CONCR

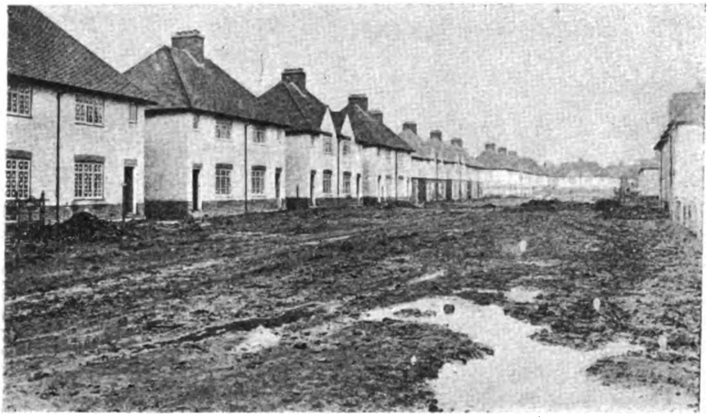


Fig. 1.—The "Road" as it Was.

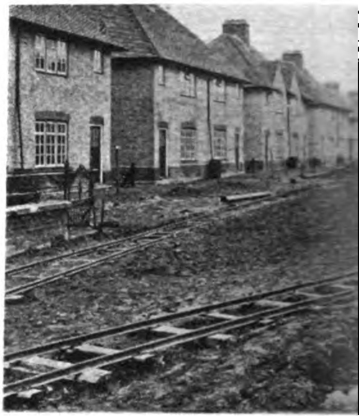


Fig. 2.—Track Laid

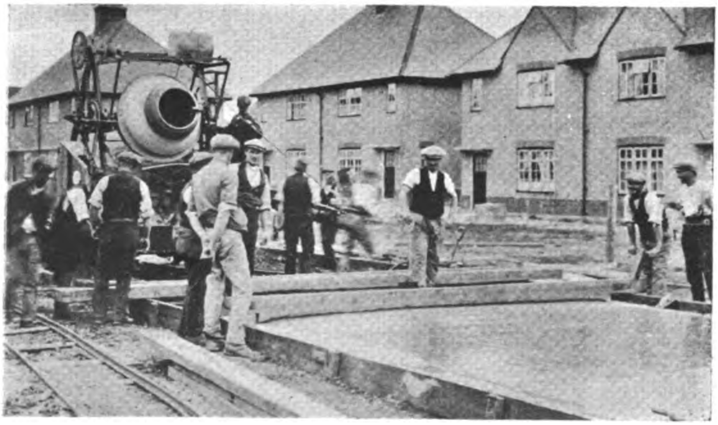


Fig. 4.—Laying and Tamping the Concrete.

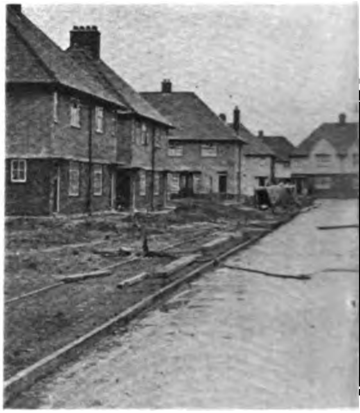


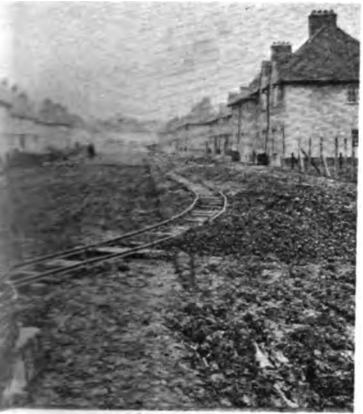
Fig. 5.—Waterin

TILBURY URBAN DISTRICT CO

Mr. S. A. Hill-Willis, A.M.I

(*See p.*

ING OF AN ETE ROAD.



Ready for Operations.

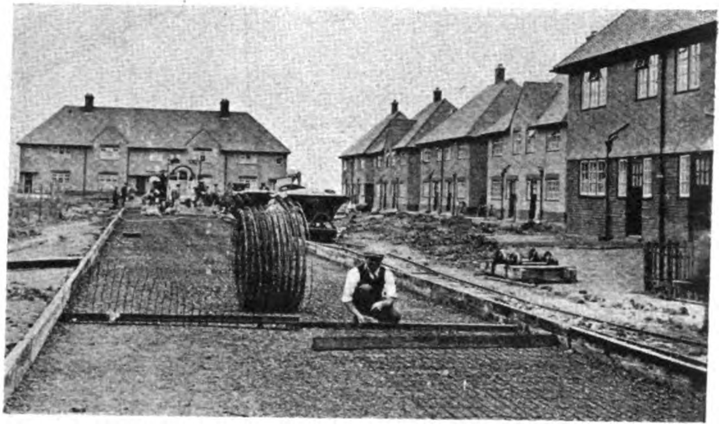
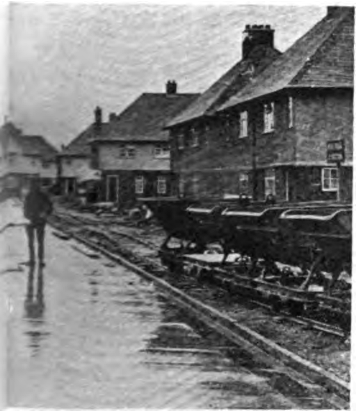


Fig. 3.—Laying Reinforcement.



the Surface.

NCIL DEVELOPMENT SCHEME.

P.L, Borough Engineer.

7.)

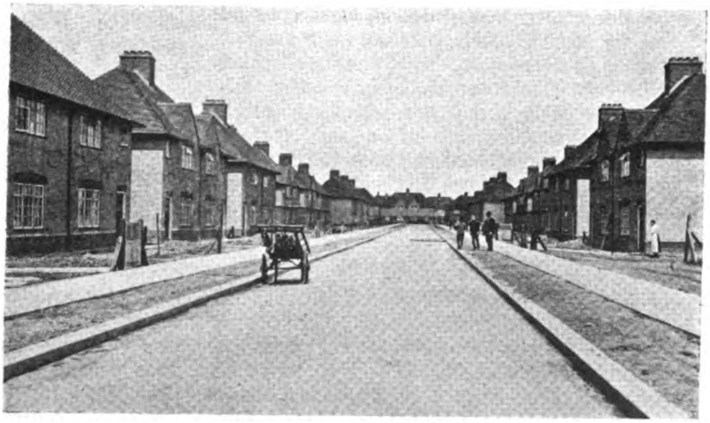


Fig. 6.—The Completed Road.

in the proportion of 3 : 1½ : 1, and the surface floated off evenly, but not smooth. The camber is 1–50.

"The most important feature of reinforced concrete roads is that the surface should be subjected to traffic for at least three months before being surface treated, and I have found it has only been necessary to tarspray and granite chip and roll twice last season. The cost of the work has worked out, including excavation to a depth of 8½ in., and including all reinforcement, concrete and labour, at 19*s*. per super yard. The road in question is subject to main-road heavy traffic, and it is now in perfect condition having had twelve months' wear. My Council are so pleased with this experimental work that I have just received instructions to do two more roads of this type.

"The great feature in reinforced concrete roads, in my opinion, is that there should be a top and bottom layer of wire of sufficient strength, tied in with shear members which keep the fabric in one true position, and it is also advisable that the sections should be laid in alternate bays so that the setting should take place before the adjoining bays are laid, and in the case of the road that was constructed interlocking joints were used."

DEPTFORD.

An all-concrete road laid in Deptford under the direction of the late Borough Engineer (Mr. F. Wilkinson) was reported on by Mr. H. Morley Lawson, the present Borough Surveyor, in November, 1920, who then stated that, with the exception of a small pot-hole, attributed to careless floating after the surface screed was removed, all the joints were perfect and showed little deterioration. This road takes all the heavy traffic to and from the Supply Reserve Depôt.

DUNFERMLINE.

After three years' service, an all-concrete road in Dunfermline, which carries a motor-'bus service and other heavy traffic, was found by the Borough Engineer to be in as good condition as when laid, and the only maintenance incurred during that period was £5 for tar-spraying. A year later (i.e., after four years' wear) Mr. R. Muir Morton, Borough Engineer, said : "The condition of the road to-day is very satisfactory, and no fault can be found with the surface. It is not anticipated that any extraordinary expense will have to be incurred thereon for a very considerable time."

EDINBURGH.

One of the oldest all-concrete roads in the country, Blackwood Crescent, Edinburgh, was laid in July, 1873. In 1920, Mr. J. Sims, City Road Surveyor, stated : "The surface of this forty-seven-year-old road is still in fair condition, and, what is very remarkable, the total cost of maintenance has been but £40 since the road was laid." The road is about a quarter of a mile long.

ESSEX.

A section of the main road from London to Colchester, at Mountnessing, was laid in 1915 with a centre strip 11 ft. wide of macadam and strips of bare concrete, 5 ft. 7 in. wide, on each side. In 1920 Mr. Percy J. Sheldon, County Surveyor, reported : "The road is in very good condition, and the cost of maintenance has been practically *nil*. The road carries very heavy traffic."

The Epping New Road at Buckhurst Hill, which carries 1,500 tons of traffic per day, has been built of bare-concrete. Five months after it was opened to traffic Mr. Sheldon reported that the cost of maintenance had been *nil*; also, that he was convinced that "there is no question as to the strength and durability of concrete roads; their success and efficiency are only questions of workmanship."

EXMOUTH.

The Marine Drive, Exmouth, was laid with an all-concrete roadway in May, 1915, and after six weeks was tar-sprayed. More than 1,000 tons of traffic passed over a strip 8 ft. wide between May and November, "and there was not the slightest sign of wear." Five years later, Mr. S. Hutton, Engineer and Surveyor to the Council, reported that "the condition of the road was very good and the cost of maintenance during the 5½ years had been less than £5," and that "concrete roads are more economical than other types."

GRAVESEND.

After five years' wear, Mr. H. T. Chapman, County Surveyor of Kent, reported that an all-concrete section at

Reinforced Concrete Works Road (6 in. thick) at Cadishead for the British Tar Products Co., Ltd.

Gravesend of the London–Dover road was practically as good as when laid; there were no signs of disintegration or cracks except at the expansion joints.

LEEDS.

Mr. E. W. Cockerlyne, Engineer to the City of Leeds, writes:—

"Recent experience in the construction of concrete roads in Leeds has led to the opinion that there is a wide field of opportunity for this type of work in municipal development schemes. Despite the fact that in the making of this form of roadway in built-up urban districts the Highways Engineer encounters local difficulties due to traffic conditions, which are not met with to such a great extent in inter-urban areas, and which do not apply in the case of new road construction over virgin ground, the concrete road must be seriously considered by any municipal authority whose aim is economy of construction and maintenance. Personally, I am watching very closely the behaviour of the concrete roads constructed in this district, and up to the present time I have been impressed by their resistance to the heavy traffic using them. Surface tarring does not appear to be a complete success, and it is not unlikely that the fine material in the concrete surface has an important bearing on this point."

NEWBURY.

Mr. S. J. Lee Vincent, Borough Engineer, reporting on a quarter-mile stretch of the main road in Newbury, laid with bare concrete in the spring of 1920, stated in December, 1920, that "after seven months of heavy traffic the surface is in a thoroughly satisfactory condition."

PADSTOW.

An all-concrete road was laid on the Fish Quay at Padstow in 1914 by the London & South-Western Railway Co. In 1920 the Chief Engineer of the Railway stated: "The present condition of the roadway is very good, and the cost of maintenance since the road was opened has been *nil.*"

SOUTHAMPTON.

An all-concrete road which takes heavy traffic at Southampton Docks was laid in 1917. In November, 1920, Mr. F. E. Wentworth-Sheilds reported: "The road has stood the traffic exceedingly well, and although a slight crack is visible at the joint of each day's work, there is no sign of deterioration here or elsewhere. What little wear there is is very even, so that there are no pot-holes or malformations of a sort likely to be intensified by further traffic. The surface is not slippery, and the road has given general satisfaction, and is still in excellent condition."

SOUTHAMPTON DOCKS.

In a recent issue of *Modern Transport,* Mr. G. S. Szlumper, Docks and Marine Engineer of the Southern Railways, states: "Realising that good roadways are the foundation of cheap and efficient transport, special attention has been paid to the roads within the docks. These roads total in length five miles, and are surfaced with various materials, tarmac being in the preponderance; we find this surface excellent, but requiring a considerable amount of maintenance and dressing, the asphalte-surfaced roads being more satisfactory in the heavy wear they get. We also have an experi-

[This photograph shows an Australian concrete road, 6-in thick, reinforced with B.R.C. fabric, from which the foundation was washed away for a length of 12 ft. and a depth of 8 ft. under half its width three weeks after the road was laid. The slab was not injured, and carried traffic until the foundation was rammed back.]

mental stretch of reinforced concrete road which wears excellently, and is, I think, the best form of road, if the necessary time can be given to ensure its proper 'ripening' before being opened for traffic."

SOUTH WALES.

After it had been subjected to traffic for about a year, Mr. S. A. Bennett, Brecon County Surveyor, said of the all-concrete section of the Brecon and Abergavenny main road : "So far the concrete road has been a complete success, having withstood the traffic and showing no signs of disintegration or cracks."

SOUTHWARK.

Over seventy all-concrete roads have been laid in the Metropolitan Borough of Southwark since 1918, to carry traffic varying from light to heavy. Mr. Arthur Harrison, M.Inst.C.E., Borough Engineer, has since reported that although weaknesses have appeared in two of the streets (owing, it is believed, to want of care in curing and unsatisfactory consolidation along the joints), all the streets opened to traffic are "in excellent condition."

TILBURY.

In 1917 a section of the main dock road at Tilbury was laid with bare concrete. Mr. S. A. Hill-Willis, Borough Engineer, reporting on this road three years later, said it had been traversed by all types of heavy traffic in and out of the Docks, and he was in every way satisfied with it; so much so that contracts had been placed by the Urban District Council for 6½ miles of similar roadway. . . . "A 50-ton trench excavator, with its near wheels shod with heavy steel studs, passed over the road, but in spite of the great weight of the machine no injury was caused to the surface."

TOTNES–PAIGNTON.

Built in 1919, an experimental section of all-concrete road between Totnes and Paignton was stated by Mr. Andrew Warren, County Surveyor (Southern Division of Devonshire), to be in November 1920, "as good to-day as when first opened, and no complaints have been received of its being slippery. The noise when steel-tyred vehicles are running over the section is no greater than when macadam was employed."

WORKS ROADS.

Altogether about 140 all-concrete roads have been laid in factories and works.

An all-concrete road 200 yds. long and 24 ft. wide laid in the works of Messrs. Joseph Baker, Sons, & Perkins, Engineers, of Willesden Junction, has been reported on as follows by Mr. S. W. Moscrip, the Company's Engineer : "The road has been in use just over three years and has cost nothing for maintenance or repairs. Heavy loads up to 13 tons in weight have been over the road. Messrs. Joseph Baker, Sons, & Perkins are thoroughly satisfied with this kind of road construction, and will certainly use it in future extensions of their works."

The approach to the Empire Paper Mills at Greenhithe was laid as an all-concrete road in 1918. Mr. D. I. MacIvor, the Company's Engineer, has since reported that "the road is in as good condition as when first opened." This road carries steel-shod steam wagons.

CONCRETE ROAD PAVING.

By ARTHUR HARRISON, M.Inst.C.E., Borough Engineer & Surveyor, Southwark.

For many decades Portland cement concrete has been regarded as essential for the foundations of roads subjected to heavy traffic, but its trial as a surfacing material for carriageways in this country is only of comparatively recent date.

Southwark was one of the first of the London Boroughs to experiment in this direction, and in 1919, at a time when demobilisation had thrown many thousands of men on the labour market, a bold attempt was made to reveal the possibilities of concrete as a road paving. A number of side streets were selected for the test, some of which carried fairly heavy traffic.

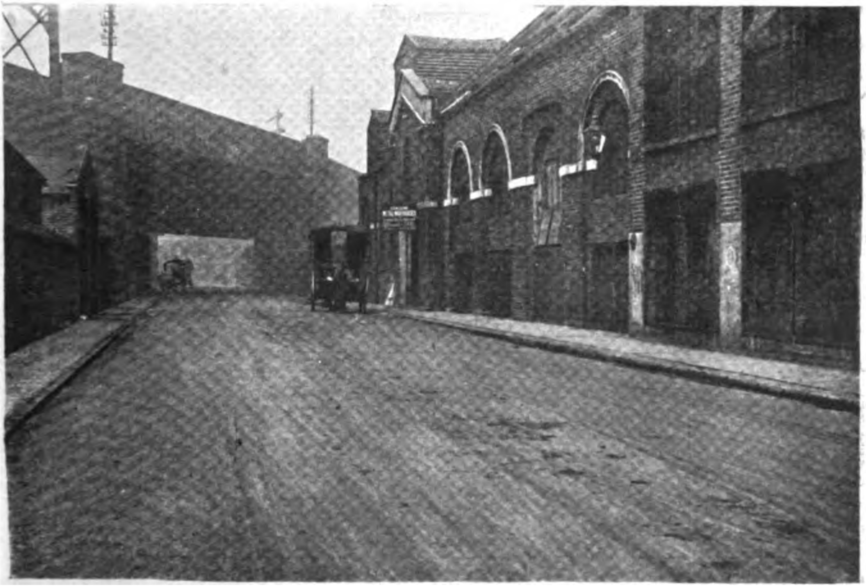
An important feature might well be mentioned at the commencement of this article that practically all the old macadam crust excavated was put to use, and, comparatively speaking, little was taken to the shoot, thus effecting a considerable saving on cost. A portable washing-machine was taken to the site of the work, where the excavation was washed and graded, care having first been taken to remove any tar or like substances from such macadam roads as had been treated in this way. The material proved to be an excellent substitute for ballast, and the aggregate used was in the proportion of 5 parts of coarse and 1 of fine (in lieu of sand) to 1 of British Standard Portland cement.

The two-course method was adopted, the lower course being 4½ in. of this mixture, whilst the surfacing or wearing coat was 1½ in. thick, consisting of 3 parts of fine crushed granite to 1 part of cement. The granite was graded in the following proportions :—

20%	¾-in. gauge.
25%	½-in. ,,
30%	¼-in. ,,
25%	⅛-in. down to dust.

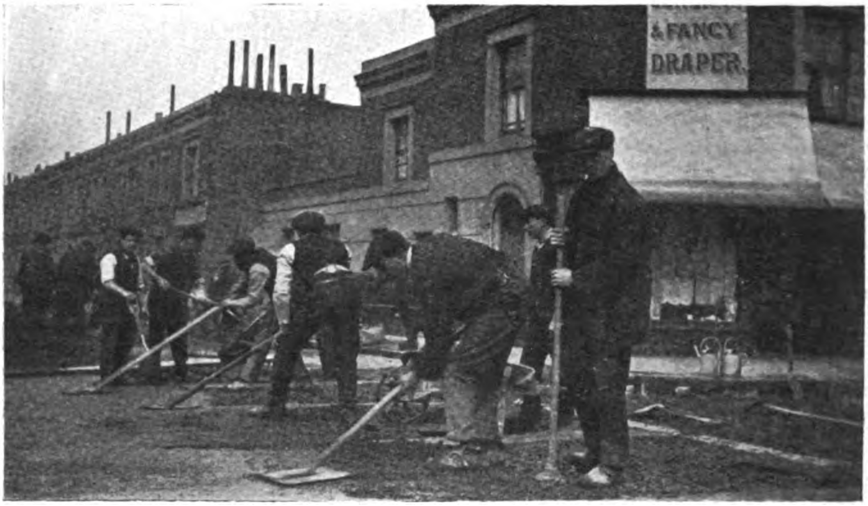
The second course was laid before the first was set in order that the whole should form one monolithic structure.

To return for a moment to deal with the reinforcement. No proprietary fabric was used throughout the work, but a simple type consisting of mild steel rods $\frac{3}{16}$ in. in diameter interlaced in mattress form to a 6 in. mesh on the site in mats of roughly 12 ft. square, successive lengths being hooked together. This

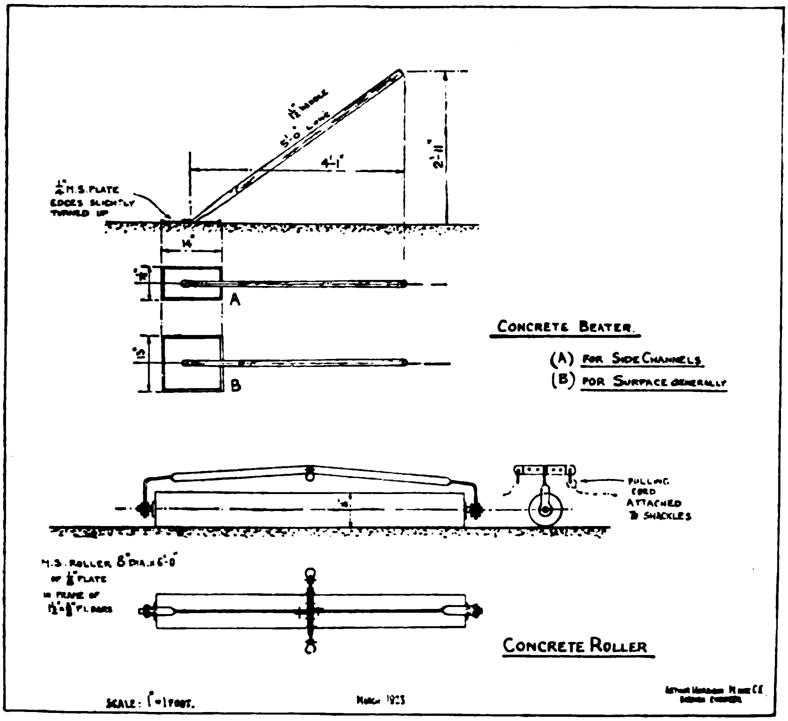


[*Mr. Arthur Harrison, Borough Engineer and Surveyor.*.

Reinforced Concrete Road at Southwark.



HAND TAMPING AND RUBBING.



APPARATUS FOR SURFACE FINISHING.

[*Mr. Arthur Harrison, Borough Engineer and Surveyor.*

REINFORCED CONCRETE ROADS AT SOUTHWARK.

fabric was laid 2 in. from the bottom, i.e., in the middle of the bottom course.

Screeds were not used, but the concrete was brought forward to proper levels and grades by boning and pegging, and finishing each day's work with a straight vertical prepared joint.

During hot weather the finished surface was protected with sheets immediately after being laid, and within twelve hours covered with a layer of 2 in. of wet sand, which was kept constantly wetted for fourteen days. This was then cleared, and the work allowed to dry for two or three days and finally coated with tar and sanded.

The roads were not re-opened for three weeks after the concrete was finished. This enforced and somewhat prolonged closing to traffic represents the one disadvantage of this type of paving.

Ordinary plant was used, except for the washing-machine before mentioned, and a special tool for finishing the surface, consisting of a metal plate 15 in. sq. to which a handle is fitted obliquely so that the tool can be operated from the face of the work. A light roller, designed for surface finishing, is illustrated on p. 274.

The line of each day's work is apparent, the work having been finished square. The body of concrete contracts slightly, and the edge tends to wear. Such joints were filled with flux grouting dusted with fine granite, and this method proved satisfactory.

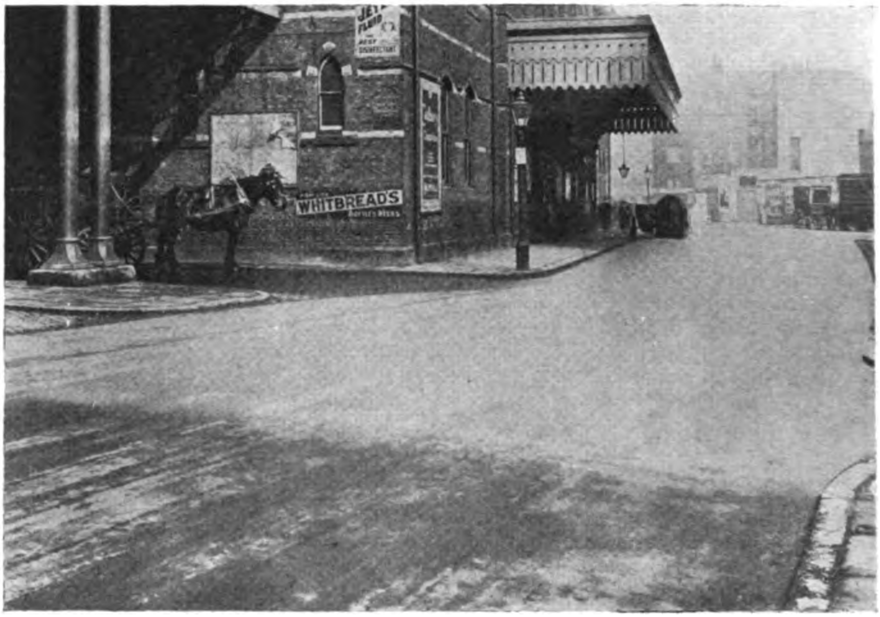
There were admittedly at first one or two failures due to inexperience, but the weaknesses revealed proved useful in showing how very necessary it is to regard the smallest details. Greater care is certainly needed in making concrete for road construction than for ordinary work. The results throughout have been most encouraging and have justified the carrying out larger schemes of conversion to concrete, principally from macadam, but in some instances wood and setts have been so replaced.

To date a total of seventy streets have been concreted—a total length of over eight miles. Many of these link up main roads and consequently carry a considerable amount of traffic.

Concrete makes a clean smooth running roadway, and our experience shows that it stands up well under test. To all appearances maintenance charges should be low.

It should be mentioned that the majority of men employed upon this work were demobilised men who had never previously performed any work of this kind

Reinforced Concrete Road at Southwark.



REINFORCED CONCRETE ROAD AT SOUTHWARK.
[The section in the foreground is tar-sprayed ; the remainder shown in the photograph is treated with silicate of soda]

and were trained by the Borough Engineer's staff.

The Ministry of Transport, Roads Department, has been specially interested, and a large number of visitors from all parts of the British Isles, and indeed from many other countries, have inspected the work whilst in progress.

Several experiments have been made on short lengths by treating the surface after fourteen days with a solution of silicate of soda. These are still under observation, but it is too early to give a definite opinion as to its value.

In conclusion, one would emphasise the necessity for exercising care, particularly in regard to the following :—

Make sure that the matrix is properly graded in order to obtain solid concrete, that is to say, without voids.

Use judgment in determining the quantity of water required for mixing, and sufficiently work the concrete to get rid of air and surplus water.

At the commencement of each day's work see that the concrete is well packed at the joints with the previous day's work.

Realise the importance of protecting and "curing" the face of the work and resting it sufficiently before allowing the paving to do duty.

CONCRETE ROADS AT TILBURY.

THE first all-concrete road was laid on a section of Dock Road, Tilbury, in 1917, by Messrs. W. & C. French, of Buckhurst Hill, from the designs and under the supervision of Mr. S. A. Hill-Willis, M.Inst.C.E.I., A.M.T.P.I., Engineer to the Tilbury Urban District Council, who, a month or two ago, said: "Through the remaining period of the war the roadway was called upon to bear a constantly increasing heavy traffic, which sometimes consisted of tanks and armoured cars, and in July, 1920, a 50-ton Parsons excavator, with the rear wheels shod with heavy studs, passed over Dock Road with serious results except to this concrete portion, which, despite the great weight of the machine, showed no trace of its passage."

In view of this experience it is perhaps natural that the Council and its Engineer should have favoured the same type for future road construction and reconstruction, and that all the roads, some 44 in number, in connection with the new development scheme and unemployed-relief works, costing over £1,000,000, should be built of concrete with the surface left bare. This scheme, which is now in course of development, comprises seven miles of new roads, varying in width from 8 ft. (for *culs-de-sac*) to 40 ft. for the roadways around the new civic square.

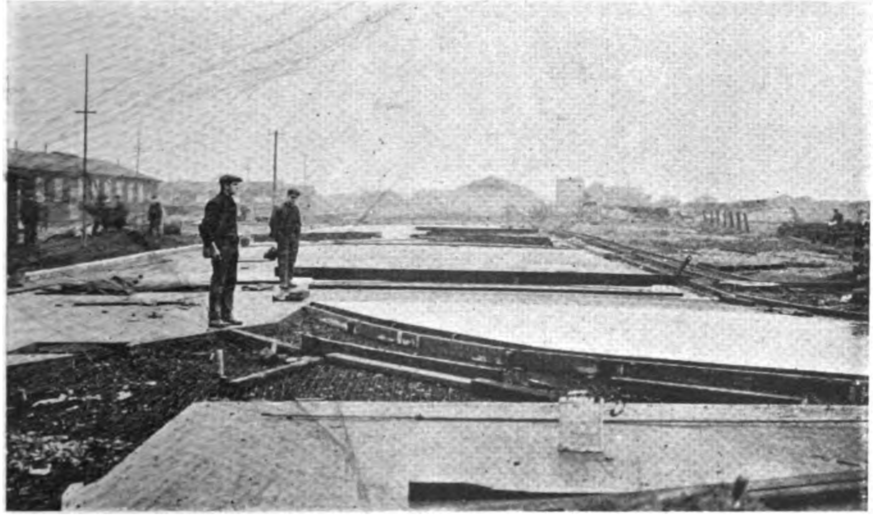
The method of construction has been specified by Mr. Hill-Willis as follows:—

The existing surface of the ground is filled or excavated, as required, to form the sub-base on which the cinders and concrete are to be laid. This sub-base is steam-rolled with a light roller to form an even surface, and a layer of cinders 3 in. in thickness is then placed over the entire surface and rolled. The finished surface thus obtained has a crossfall of 1 in 50, 1 in 40, or 1 in 36, as required.

The concrete for the lower course consists of 4 parts ballast and 1 part Portland cement. The thickness of the lower course is 4½ in. The ballast used in the lower course is approved material free from clay, peat, ligneous, earthy, or other deleterious matter, etc., not exceeding 2 in. gauge, and having sufficient sand and fine material to fill the interstices. If the voids exceed 20 per cent., sand or stone is added to reduce the voids to not more than 20 per cent.

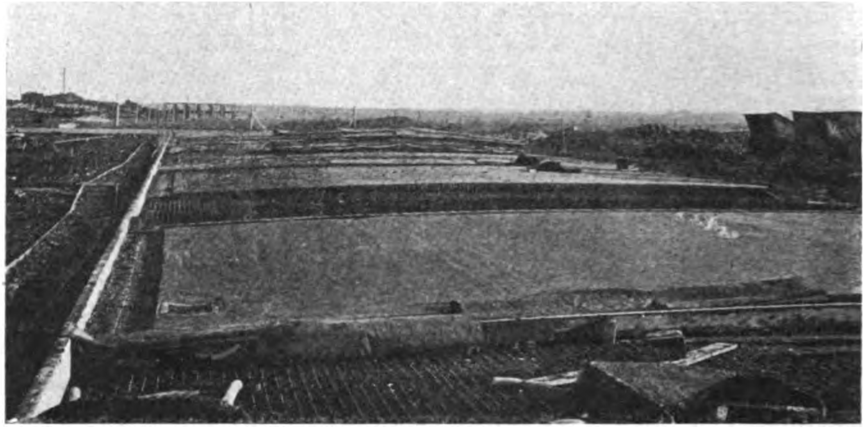
The top course is 1½ in. in thickness, and consists of 2 parts best granite chippings graded from ⅛ in. to ½ in., 1 part sand, and 1 part Portland cement.

The concrete is mixed in batch mixers



[*Mr. S. A. Hill-Willis, Borough Engineer.*

ALL-CONCRETE ROAD (40 FT. WIDE) AROUND THE CITY SQUARE, TILBURY.



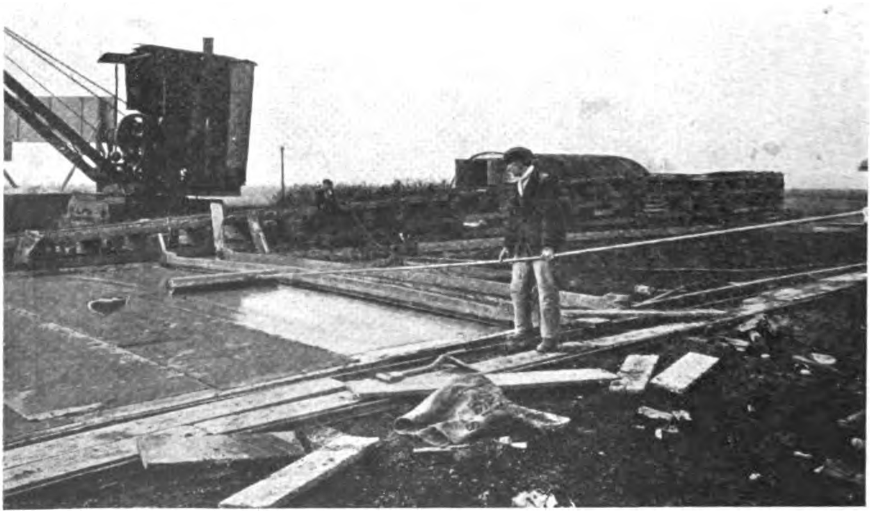
All-Concrete Road (40 ft. wide) around the City Square, Tilbury.

separate mixers being used to enable the placing of the top course to be carried along simultaneously with the lower bed. In no case is a period of more than 20 minutes allowed to elapse between the deposition of successive batches.

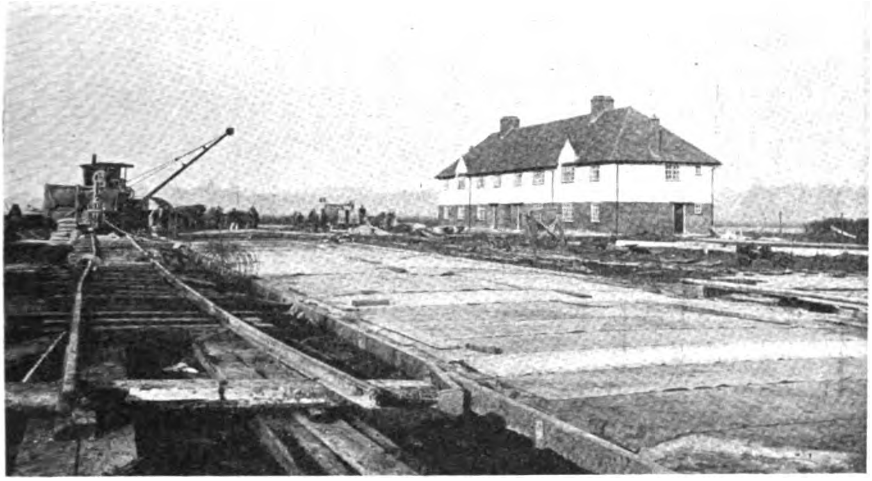
The concrete is deposited immediately it is mixed, wood forms, against which the concrete is finished, being placed at each edge of the road. These forms consist of 2-in. planks nailed to wooden spikes driven into the ground at the back of the plank. The top edge of the planks finishes at a level to conform with the finished surface of the road, and the planks are used as supports for the templates for finishing off the road surface.

The concrete is reinforced with B.R.C. steel-wire mesh reinforcement, placed 1½ in. from the bottom. The reinforcement has 4-in. side and 18-in. end laps.

At the expiration of each day's concreting the concrete is finished with a rough vertical edge, and a strip of reinforcement 3 ft. long, the full width of the road, is bedded a distance of 1 ft. 6 in.



All-Concrete Road at Tilbury, showing Hand-Tamper and Method of Rolling the Surface.



All-Concrete Arterial Road at Tilbury, showing Side-forms and Covering of Canvas.

in the completed concrete at a depth of 1½ in. below the top surface, and 1 ft. 6 in. is left projecting to bond into the new concrete when concreting is resumed.

As soon as the top-course concrete is laid the surface is struck off by means of a template resting on the wood forms at each side of the road, the template being moved over the surface with a combined longitudinal and transverse motion. Any excess of material accumulating in front of the template is uniformly distributed over the surface of the new road. The excess water is squeezed out of the concrete by rolling the surface from side to side of the road with a hollow roller about 5 ft. long by 8 in. diameter, made from sheet steel, weight about 80 lb., and followed up with a rubber or canvas belt, about 8 in. wide, worked by a man at each side of the road and drawn across the surface of the concrete with a slight forward motion to give a gritty appearance. After the concrete has assumed its initial set no tamping is permitted.

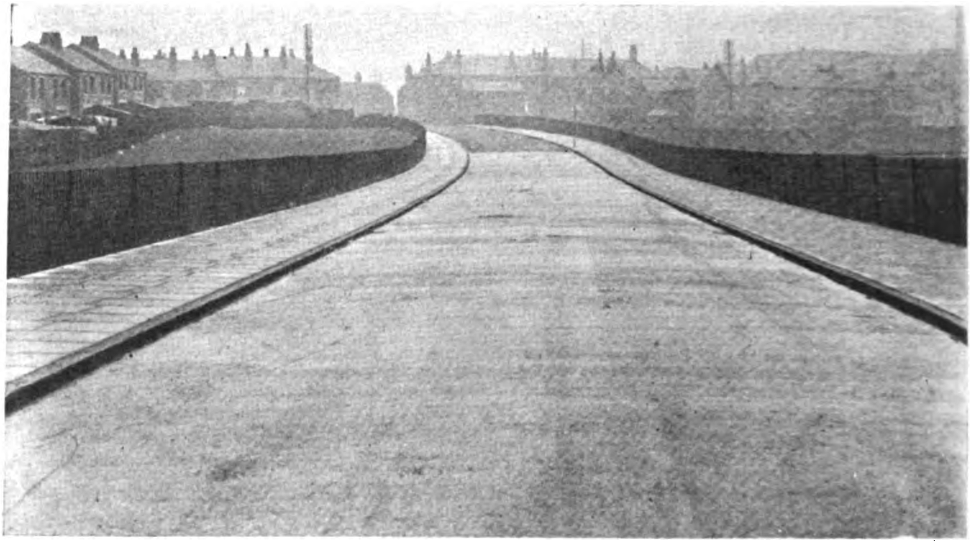
Wood planks, 12 in. by 1½ in., are temporarily laid on the bottom-course concrete to reserve the space for the kerb, and on these being removed the 12-in. by 6-in. flat kerbs are immediately bedded in position in cement mortar.

When the concrete has been laid the surface is covered over with tarpaulin until it has set sufficiently for a man to walk upon its surface without disturbing it: it is then fenced in and kept free from all traffic for not less than 28 days.

Where a concrete road joins or crosses another concrete road at an angle, neither the concrete nor the reinforcement of the two roads is joined together, but a row of 4-in. granite setts is laid to form the surface of each concrete road along the line of the joint. The concrete underneath the setts is 6 in. in thickness, and this extra thickness is carried back for a distance of 12 in. from the edge of the joint. The reinforcement under the setts remains at 4½ in. from the top surface of the road. The granite setts are laid at the same time as the concrete, and grouted with fine concrete.

At the terminations of concrete roadways and junctions of concrete and other roadways, there is provided a double row of 4-in. granite setts on Portland cement concrete 6 in. in thickness.

The contractors for the development and relief scheme are Messrs. Trollope and Colls, Ltd., of 15 West Halkin Street, Belgrave Square, S.W.1, who succeeded Mr. T. W. Robinson and commenced work for the Council in 1920.



[*Mr. G. Hayes, Borough Surveyor.*

Reinforced Concrete Road at Westhoughton, for the Westhoughton Urban District Council.

[This road is 6 in. thick, laid in two courses. The bottom course, 4 in. thick, is composed of local stone, pit sand and Portland cement in the proportion of 4 : 2 : 1 ; the top course, 2 in. thick, consists of a 3 : 1½ : 1 mix of ½ in. granite chippings, granite sand ($\frac{1}{16}$th dust), and Portland cement.]

[*Mr. F. Sadler, Borough Surveyor.*

Western Avenue : Reinforced Concrete Road at Acton.

CONCRETE ROADS AT MIDDLESBROUGH.

By S. E. BURGESS, M.Inst.C.E., Borough Engineer.

HAVING regard to the distress prevailing at Middlesbrough by reason of the closing down of the blast furnaces and the great number of men thrown out of employment in consequence, after conferring with the Government the Corporation of Middlesbrough has had many schemes approved, the majority of which are completed or in hand.

After some roads had been carried out, the Borough Engineer was instructed by the Special Committee of the Town Council to advise as to further works that could be put into operation on which the maximum of labour could be utilized, and on considering the matter he reported that having regard to the great number of roads in the centre of the town that were in need of repair—which to some extent had been neglected during the economy exercised during the war years—reinforced concrete carriage-ways to streets and roads should be carried out.

In the first report on the matter it was suggested that one street or road in each of the eleven Wards into which the Borough is divided should be put in hand, so that road reconstructional work could be fairly and evenly distributed over the whole of the Borough, and these roads were recommended for reconstruction in reinforced concrete. The Committee approved of the report, and authorised the work to be gone on with, conditional that an application were made to and approved by the Unemployment Grants Committee for a substantial contribution for the carrying out of the suggested reconstruction by putting on available unemployed labour; and of a total estimated cost of £80,000, of which £40,000 was for labour, the Unemployment Grants Committee made a grant of 60 per cent. on the labour estimate, and approved the work with a grant of £24,000. The Ministry of Health, after conferring with the Ministry of Transport, has sanctioned a loan for the remainder of the total estimated cost.

The reinforced concrete roads completed last winter season under this scheme, seventeen in number, were as follows :—

Gilkes Street	£4,954
Dunning Street	1,408
Marsh Street	5,233
Snowdon Road	7,360
Nelson Street	1,437
Southfield Road	9,439
Diamond Road	3,386
Ayresome Street	5,636
Woodside Street	1,603
Wilton Street	3,677
Bright Street	4,650
Abingdon Road	3,750
James Street, N.O.	3,900
Charles Street, N.O.	3,900
Park Lane	6,750
Worcester Street	3,840
Park Road South	8,625
Total	£79,548

[*Mr. S. E. Burgess, Borough Engineer and Surveyor.*

SNOWDON ROAD: REINFORCED CONCRETE ROAD AT MIDDLESBROUGH.

[*Mr. S. E. Burgess, Borough Engineer and Surveyor.*

SOUTHFIELD ROAD: REINFORCED CONCRETE ROAD AT MIDDLESBROUGH.

It is found in actual practice that some 45 per cent. to 50 per cent. of the total cost is a labour charge.

The reinforced concrete roads have been carried out most satisfactorily (except in odd cases) by the employment of unskilled labour (all and sundry are put on as sent forward by the Employment Exchange on application for the requisite number of men that are required on the specific days at the different roads that are in process of reconstruction).

The concrete road works carried out under the schemes for relief of unemployment this winter season are twelve in number, as follows:—

Gosford Street	£1,821
Brougham Street	1,662
Richmond Street	1,864
Garbutt Street	1,116
Washington Street	3,495
Vaughan Street	3,442
Johnson Street	2,464
St. Barnabas Road	7,160
Garden Street	2,546
Wellington Street	2,880
Ayresome Green Lane	11,516
Roman Road	6,276
Total	£46,236

The roads carried out have been reinforced with Walker-Weston reinforcement (i.e., compound double-layer steel fabric—10 lb. of steel reinforcement is used per sq. yd.), B.R.C. fabric, and expanded metal ("Expamet"). The cost of the work has come out (all in) at something approaching 20*s.* per square yard.

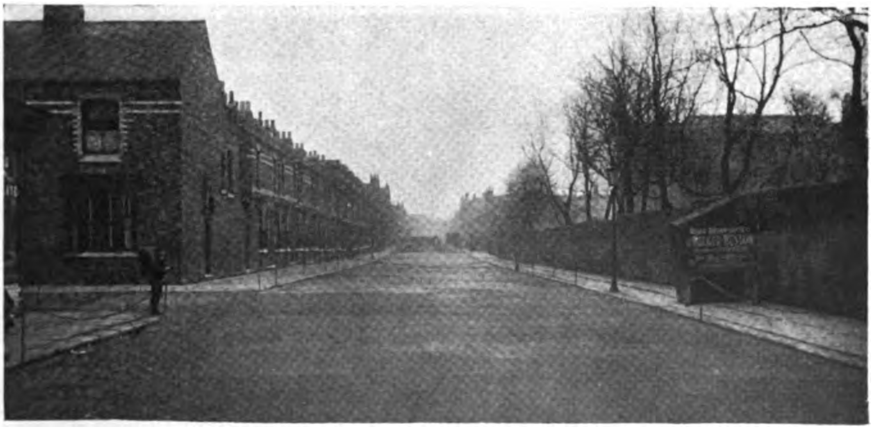
The Corporation was pleased with the work undertaken last winter in so far that the remaking and rebuilding of the Stockton main road was put in hand, and the work is now completed. This is a classified No. 1 main road connecting Middlesbrough with Thornaby and Stockton. Having regard to the number of large works in the area, this road is subjected to a considerable amount of traffic, some of it being of a very heavy nature. The length within the Borough of Middlesbrough is about 1 mile, and the old widths averaged about 30 ft. for the carriage-way, which included a single track of tramways with usual passing places. The levels of the greater portion of the length varied from 12 to 14 ft. above O.D., or only 3 ft. 6 in. to 5 ft. 6 in. above high water (ordinary spring tides), and in cases of heavy rain storms it was liable to flooding. The subsoil is soft clay and peat in parts, and the surface previously did not remain satisfactory very long after being repaired.

The road has been widened to a width for the carriage-way of 40 ft. between kerbs, and the levels of the road have been raised 12 in., 18 in. and in places 27 in., and a double track of tramways laid. The concrete roadway, 8 in. in thickness, has been reinforced with Walker-Weston interlocked framework fabric, 5 in. in depth, which has been put on the top of the old road. The surface is paved on fine slag and cement bedding, with 4 in. deep Whinstone setts at the sides of the road and 5 in. deep similar setts between the double line of tramways. The reinforcement is continuous bonded from kerb to kerb, and in this way a continuous concrete raft or girder is formed on top of which the permanent-way of the tramways is constructed. The cost of the work is £68,000.

All the reinforced concrete carriageways that have been carried out and are in hand have been to the following specification :

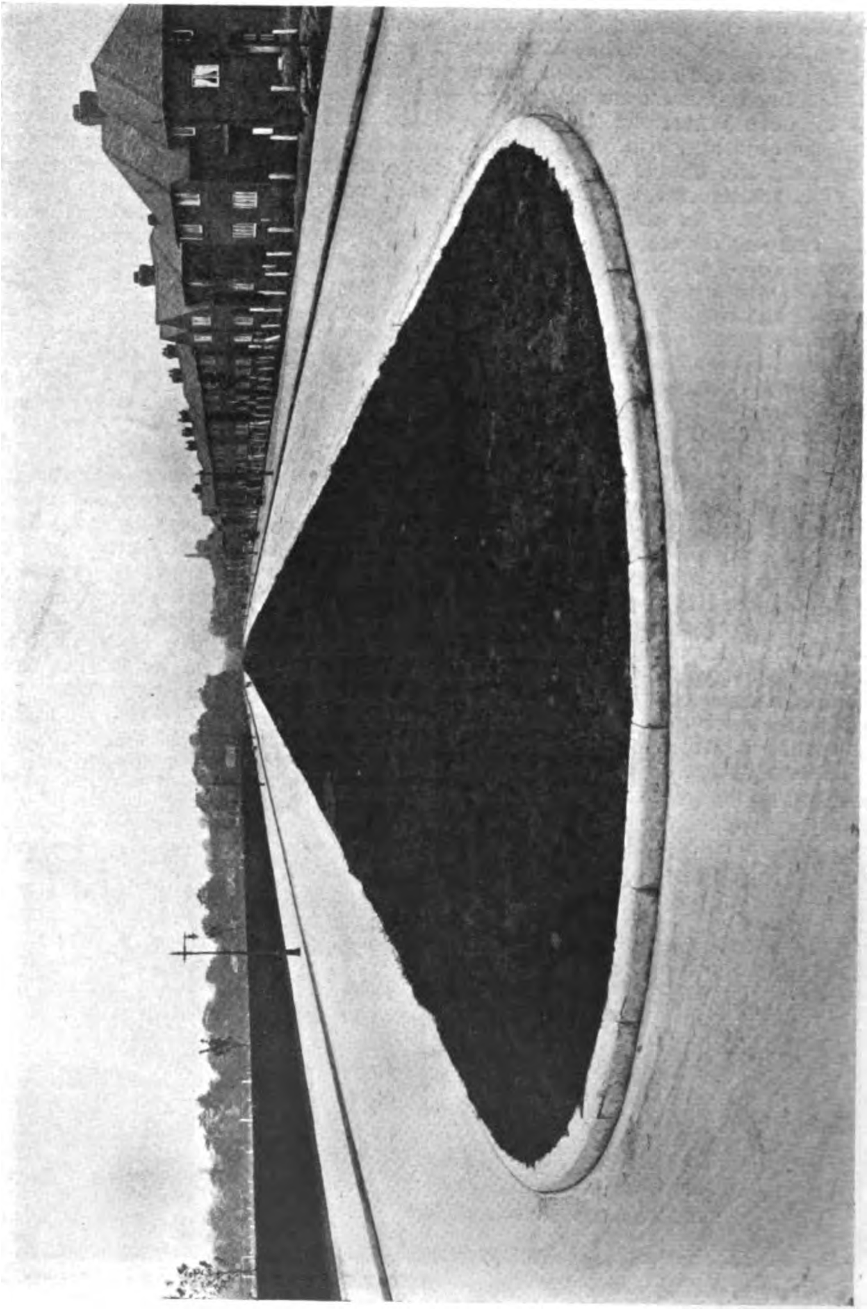
" Concrete to be 6 in. to 8 in. thick, reinforced with Walker-Weston double layer, or B.R.C. No. 9, ' Expamet ' or other approved reinforcement, laid from 1 in. to 2 in. above the bottom of the concrete. Surface to be finished with a camber of 1 in 50. The concrete gauged 4½ to 1, composed of 3 parts of Whinstone 1⅛ in. to 1¼ in. gauge, 1½ parts coarse pit sand, 1 part of Portland cement, all measured by volume. Concrete all hand mixed turned over on a banker or mixing-board twice dry and twice wet; water sprinkled from a rose-watering-can ; mixture as dry as possible so long as it is workable. Ground before concrete is laid to be thoroughly soaked, if necessary. If work is left overnight, finish with straight vertical edge across the road, the reinforcement being carried beyond the day's work joint in order to bond into the next day's work. Concrete laid in solid mass in suitable layers one on top of the other while the mixture is green and struck off by wooden template and finished roughly with wooden floats. The concrete when left overnight to be covered with tarpaulins, and next day with sand which is kept wet for a fortnight afterwards. Later, when weather is suitable, concrete surface painted with a mixture of hot pitch and tar (the surface of the concrete being swept beforehand), and finally top dressed with Whinstone chippings and annealed slag chippings."

The old ordinary macadam roadways have been done away with, the old material excavated to the required depths, and reinforced concrete carriage-ways carried out. It is anticipated that no cracking either transversely or longitudinally will be experienced, and up to date no difficulty has been found in this way. Endeavours have been made to construct the concrete roads that have been carried out in such a manner that cracking will not be experienced, and if this can be maintained then true economy both in the design and construction is guaranteed.



[*Mr. S. E. Burgess, Borough Engineer and Surveyor.*

ABINGDON ROAD : REINFORCED CONCRETE ROAD AT MIDDLESBROUGH.

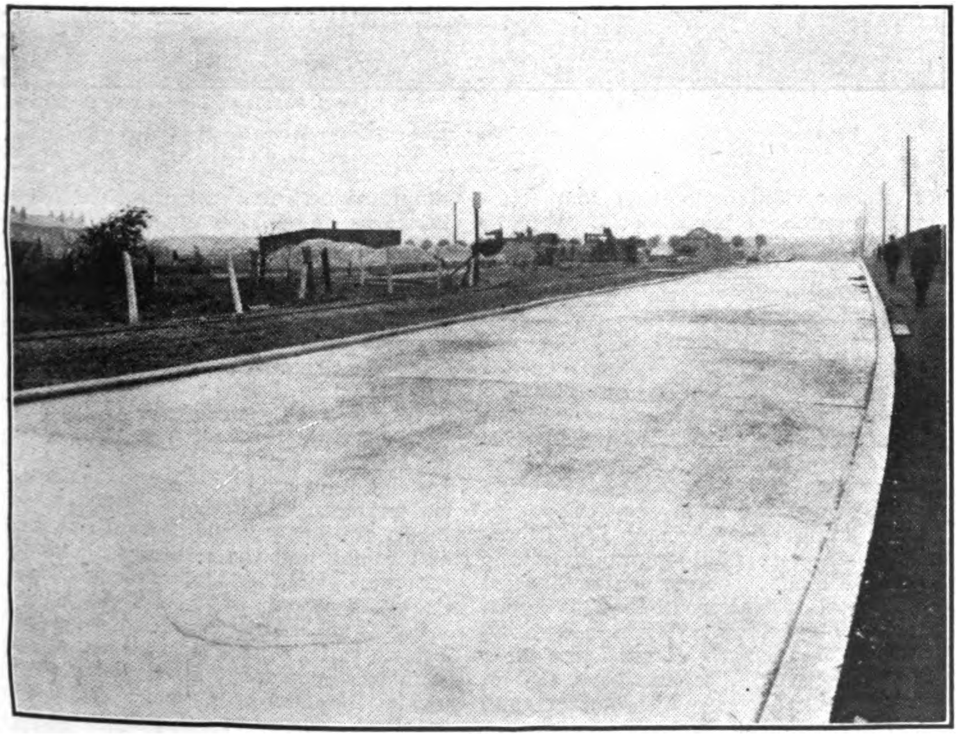


CONCRETE ROAD FROM PRINCESS ROAD TO WILBRAHAM ROAD, MANCHESTER.
[One of Twenty Reinforced Concrete Roads in Manchester.]

REINFORCED CONCRETE ARTERIAL ROADS AT MANCHESTER.

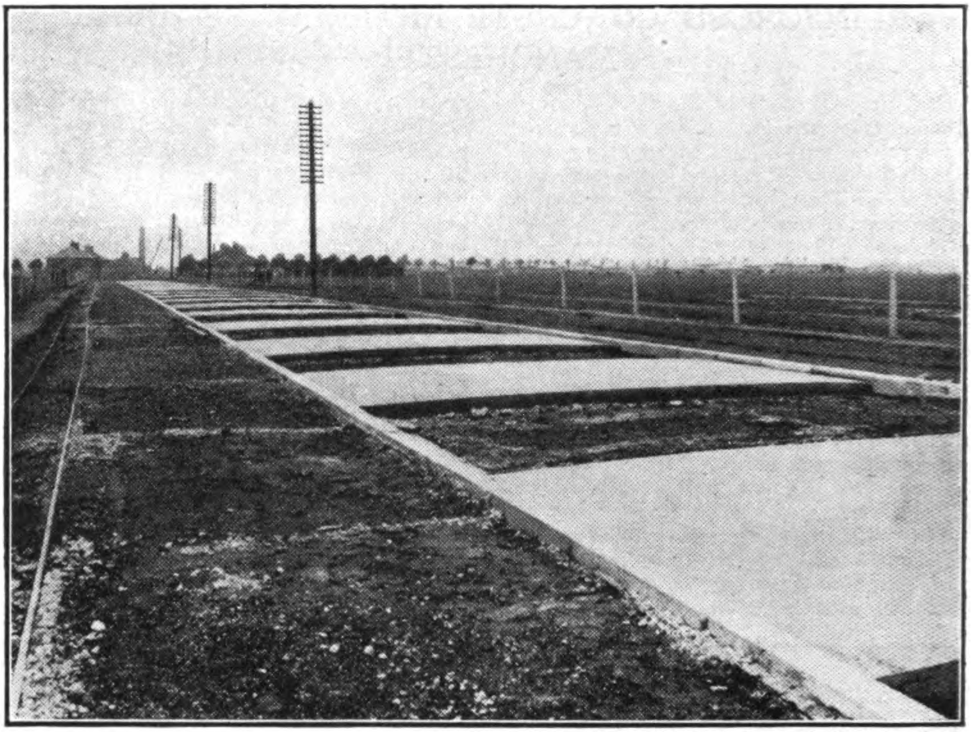
THE arterial roads in the town-planning schemes of the Manchester Corporation, many of which are now being built entirely of concrete and reinforced concrete with no top dressing, were fully described in our issue for November last. The new main roads are being constructed of widths of 80 ft. and 100 ft. between fences with a view to possible future requirements.

Some time previous to the commencement of these roads the Corporation laid down a concrete road in another part of the city in six experimental lengths, and as observations of this road indicated that a concrete surface would be successful in the case of the new road, this method of construction was accordingly adopted. The roads are not all reinforced, as reinforcement has only been used in the case of bad foundations or on embankments or where trenches have recently been opened across the roads to provide connections for the services from one footpath to the other. All the connections required by the various services (gas, electric lighting, water and sewers) were laid before the work of concreting commenced and reinforcement was used over the trenches. The method of constructing the concrete carriage-way is the same on all roads. Where there is a sound foundation capable of supporting from 30 cwt. to 2 tons per sq. ft., which is generally the case, the method of constructing the carriage-way is as follows: 3 in. of ashes or clinkers are rolled and consolidated on the natural foundation to form a bed for the concrete. On this bed, after well watering, the concrete is directly placed. The total thickness of the concrete is 7 in., laid in two courses, the bottom course being 5 in. in thickness and the top course 2 in. The proportions



[*Mr. J. B. L. Meek, Engineer to the Corporation.*

REINFORCED CONCRETE ARTERIAL ROAD AT MANCHESTER.



[*Mr. J. B. L. Meek, Engineer to the Corporation.*

Reinforced Concrete Arterial Road at Manchester: Showing Alternate Bay Construction.

of concrete for the bottom course are approximately 5 parts stone, 2½ parts sand, and 1 part Portland cement, while for the top course the proportions are 2 parts stone, 1 part sand, and 1 part Portland cement. The voids in the stone are frequently measured and the proportions adjusted to obtain as dense a mixture as possible. The stone in the bottom course is a good hard stone from 1½ in. to ½ in. gauge; the sand for this course is clean and free from organic matter but not very sharp. For the upper course the stone consists of approved granite from ¾ in. to ½ in. gauge, and good sharp sand or granite grit is used. All the concrete is hand-mixed and deposited in alternate bays, as shown in the illustration. When concreting started on a length of road one gang proceeded to lay alternate bays, and in from ten to fourteen days another gang followed and filled in the bays not completed. In laying the concrete the bottom and top courses are laid practically simultaneously in order that the whole thickness of the concrete might set together in one homogeneous mass; and when once concreting has been commenced on a bay no cessation of work is allowed until the work is completed. The top surface of the concrete is finished off to the required levels and falls with a wooden or steel-shod template.

Where cross trenches have recently been excavated in the carriage-way foundation, single-mesh reinforcement is used. In the reinforced concrete surface on embankments and bridge approaches, the banks are first consolidated by watering and rolling and the reinforced concrete structure placed on the well-rolled foundation of ashes. In these sections provision is made for tension in the top and bottom surfaces, and also for shear, and Walker-Weston patent double-layer interlocking reinforcement, which has been specially designed to make provision for these stresses, is being used. On these sections the depth of the concrete is 9 in., and the carriage-way is extended 1 ft. 6 in. under the kerb at each side of the road in order to give support to traffic travelling close to the kerb. As with the sections on more solid foundations, these portions are constructed in bays. Concrete is first laid to a depth of 1 in., and on this is placed the reinforcement, which is 6 in. deep. Concrete is well-worked round the reinforcement, which is covered with concrete to a depth of 2 in. between its upper surface and the surface of the road.

One length of 80-ft. roadway passes for a distance of a mile through a peat bog varying in depth from 5 ft. to 10 ft. below formation level. Here the peat has been taken out for a depth of 1 ft. 6 in. and the space filled with ashes and clinkers, rolled. On this surface the concrete slabs reinforced with Walker-Weston reinforcement are built, each slab being continued for 2 ft. 6 in. beneath the kerb on each side. The thickness of these slabs is 12 in. at the centre and 9 in. at the edges, with reinforcement 9 in. deep at the centre and 6 in. deep at the sides.

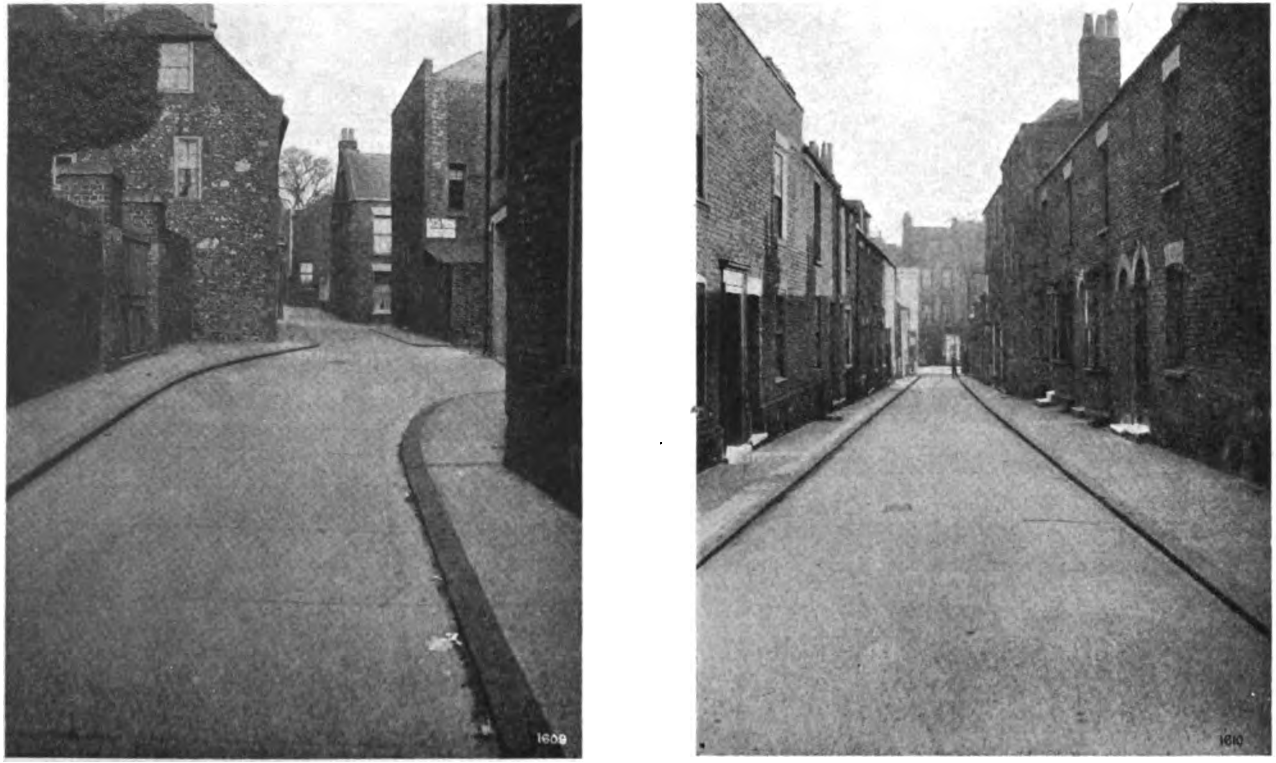
The kerbs and fence posts used on these roads are all made of concrete in the Corporation's depot adjoining the site, the posts being reinforced with four ½-in. steel bars. The paving flags for the footpaths are all of concrete, manufactured where required on the finished road surfaces.

Some pieces cut out of the road for the full depth have been subjected to tests, which showed that after being kept fourteen days in a dry room and afterwards immersed for eighteen hours in water, the absorption was approximately 3 per cent.

Some sections of these arterial roads have been in use for over a year, and other sections of all-concrete roads in Manchester have been open to traffic for two years; in neither case, states Mr. J. B. L. Meek, M.Inst.C.E. (Engineer to the Corporation), does the surface show any serious sign of wear.

[The illustrations with this article are from *Modern Transport*.]

Reinforced Concrete Road at Southampton.
[This road is laid on ground made up of refuse, and carries lorries up to 15 tons.]



Twenty-year-old Reinforced Concrete Roads at Ramsgate.

[These photographs, for which we are indebted to the Indented Bar Engineering Co., Ltd., show two reinforced concrete roads at Ramsgate after twenty years' wear. The Indented Bar Co. state: "There is no record of the mixture of concrete used in the construction of these roads, but except for the cost of repairs after the laying of pipes, there has been no maintenance costs whatever during the whole period.]

THE CONSTRUCTION OF A CONCRETE ROAD.

So much depends on the nature of the soil and the traffic to be carried that it is impossible to lay down any hard and fast method for the construction of an all-concrete road. The following notes, however, give the general practice now followed in this country :—

MATERIALS.

The cement is such an important factor that it is essential that it should comply with the requirements of the British Standard Specification, and should be slow-setting cement.

Care should be taken in the selection of the aggregates. No natural deposits of sand and gravel should be used without washing, screening, and grading. If crushed stone is used it should be screened, graded, and, if necessary, washed. No aggregate should be used which is not hard and tough, or which is laminated, and upon crushing breaks down into flat or elongated particles. Soft or porous materials, such as broken brick, breeze, etc., should be prohibited. All aggregates used must be clean and free from clay, dust, vegetable and other foreign matter. Care must be taken that the aggregate is not contaminated with mud, etc., after delivery to the site of the work.

For one-course roads no aggregate should be used which will not pass through a screen having square openings of 1 in., but for two-course roads the bottom course may contain aggregate the largest stones in which will pass through a 1½-in. square opening. None of the coarse material should pass through a ¼-in. square opening. The grading from the maximum to the minimum sizes should be regular, and no material should be used which contains a large proportion of stones of approximately one size.

Sand or fine material should pass through a ¼-in. square opening, but not more than 10 per cent. by weight should pass a sieve having 50 meshes per lin. in. The grading from the maximum to the minimum sizes should be regular, and no material should be used which contains a large proportion of particles of approximately one size.

The average compression resistance of not less than three test pieces of the concrete should not be less than will comply with the following formulæ :—

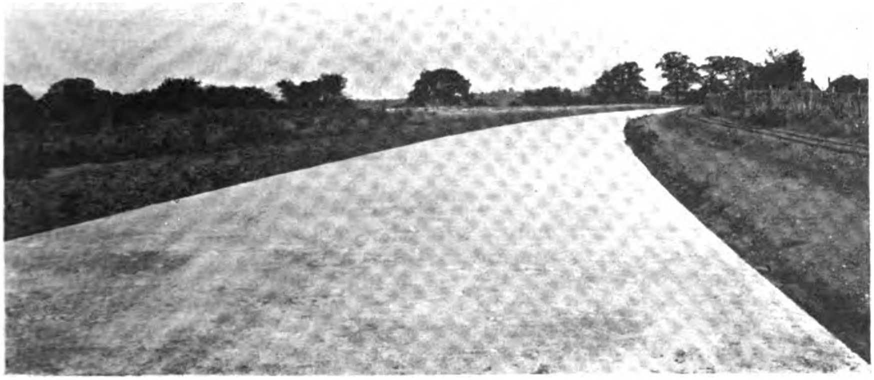
When 4 weeks old :—$C_1 = 2{,}800 - 200\,V$, and

When 13 weeks old :—$C_3 = 3{,}600 - 200\,V$, where

C_1 and C_3 = compression resistance in lb. per sq. in.

V = volume of sand and coarse material per volume of cement.

For determining the compression resistance, tests should be made on cubes or cylinders of not less than 6 in. each way which as far as possible conform to the



[*Mr. Alfred Dryland, County Engineer of Middlesex ;*
Mr. F. Wilkinson, Engineer to the Willesden District Council.

REINFORCED CONCRETE ARTERIAL ROAD AT WILLESDEN.

SMOOTHING SURFACE AT JOINTS WITH 6-FT. STRAIGHT-EDGE.

OBTAINING A UNIFORM SURFACE BY LIGHTLY BRUSHING WITH SOFT BRUSH BEFORE CONCRETE HAS FINALLY SET.

AMERICAN ROAD FINISHING METHODS.

conditions that will obtain in the actual execution of the work.

The water must be fresh and clean.

Joint filler, if used, must be an elastic waterproof material, which will not lose these properties under extremes of weather conditions.

PREPARATION OF SUBSOIL.

The preparation of the foundation will necessarily vary with local conditions. Any necessary embankments or fills should be thoroughly consolidated, so that there is no possibility of settlement at any point. Any soft or weak places must be excavated and filled with hard stone or other suitable material, so as to obtain solidity equal to the remainder of the surface. The surface should be finally rolled to the required contour with a roller of not less than 10 tons in weight. The surface thus prepared must be regular, either flat or with a slight cross fall. Immediately before the concrete is placed, all foreign matter must be removed from the prepared surface, which must then be thoroughly watered.

CONCRETE.

For one-course roads not more than 5 parts of coarse and fine aggregate to one of cement should be used. Where severe traffic conditions are likely to be met, the concrete for a one-course road should be in the proportions of 1 part of cement to 1½ parts of fine aggregate and 3 parts of coarse aggregate. For two-course roads more than 7 in. thick not more than 8 parts of coarse and fine aggregate to one of cement should be used for the lower course, and not more than 3 parts of fine aggregate to one of cement for the surface course. When the total thickness of a two-course road is less than 7 in., the proportions of aggregate and cement should not be less than 5 to 1.

The concrete should preferably be mixed in a batch mixer, and mixing must continue until the ingredients are homogeneous and plastic throughout. The drum must be completely emptied after mixing each batch. The quantity of water should be such as to secure a plastic mixture which can be easily worked, so that only light tamping will be necessary to consolidate when placed in position. Care must be taken to prevent an excessive amount of water being used, and the concrete should not be so sloppy as to cause a separation of the coarse aggregate from the mortar during handling and laying. Any concrete which has partially set before being placed in position should not be used.

The concrete should be deposited over the whole width of the road at one and the same time, except in cases where it is impossible to divert the traffic for the time being. Where there is no kerb, the sides of the road at the end of each day's work should be supported by a wooden or metal form to resist straining out of shape under the pressure of the concrete. All mortar and dirt should be removed from the forms which have been previously used, and the forms should be greased or oiled before any concrete is deposited against them. The side forms should remain in place until the concrete is set sufficiently hard to permit of their removal without damage to the edges. In removing the form at the end of the previous day's work, in order to resume operations the next day, very special care must be taken to see that the concrete is not damaged, and in beginning the deposition of concrete special care must be taken that the previous day's work is not disturbed or damaged in any way. If desired, joints filled with mastic material may be provided between each day's work, but should not exceed ¼ in. in thickness.

Under average conditions, a total thickness of about 6 in. will probably be sufficient. The surface course of two-course roads should be approximately 2 in. thick. When a two-course road is being laid the upper course or wearing surface must be spread on the lower course immediately after the latter is deposited and before it has begun to set.

Immediately after being placed in position the concrete must be struck off to approximate grade and camber and then thoroughly consolidated to eliminate all voids and surface moisture. This consolidation is effected by tamping the surface of the concrete, either mechanically or by hand, or it may be done by rolling. In no event should any tamping device be used which necessitates a penetration below the concrete surface. The finished surface should have a cross fall sufficient only to ensure that the surface water will pass off the roadway quickly, and should not vary more than ¼ in. from true shape. It will be found that, at the most, a cross fall of 1 in 60 will be sufficient.

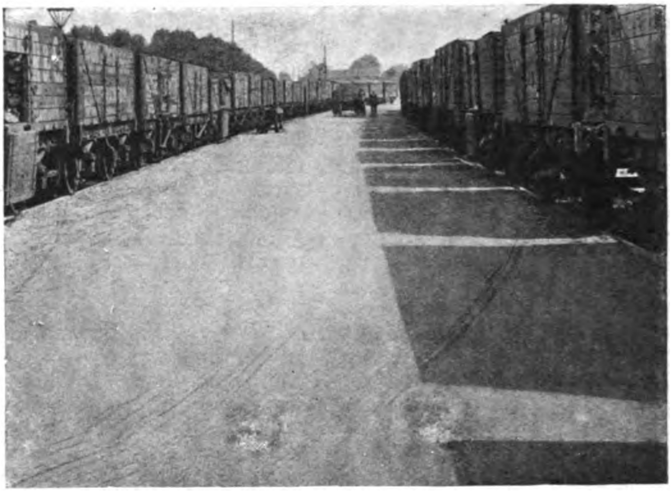
In cool weather no cessation of work of more than an hour's duration should be permitted, except at the end of a completed bay. In hot weather the work must, if possible, be continuous.

In cases where it is impossible to divert the traffic and the concrete has to be laid on one-half of the road at a time, the edge of the concrete in the centre of the road should be left with a rough vertical edge, and immediately before filling in the concrete for the second half this edge should be chipped back vertically to secure a good bond. The concrete may be reinforced with steel reinforcement at the discretion of the surveyor.

The surface of the concrete should be sprayed with water as soon as it has sufficiently hardened to withstand pit-

REINFORCED CONCRETE ROAD AT THE DERBY WORKS OF MESSRS. ROLLS-ROYCE, LTD.

[This road is 40 ft. long and 7 in. thick; the kerbs are of concrete cast *in situ*.]



REINFORCED CONCRETE GOODS YARD AT EAST CROYDON (L.B. & S.C. RLY.).

[The sidings at this Goods Yard are chiefly used for the discharge of coal, and the concrete roads, therefore, carry heavy traffic, including steam wagons.]

ting, and kept wet until covered, within 24 hours, with clay, earth, or other material, which must be kept thoroughly wetted for a period of at least 10 days. When sunshine, a drying wind, or other conditions make it desirable the freshly laid concrete should be protected by canvas laid on a wooden framing or other covering until set sufficiently to be watered and protected as prescribed.

The road should not be opened to traffic until at least 21 days after it is laid, and when the weather is cool or wet this period should be increased. Where the road is constructed in two halves the traffic should not be concentrated on to the first half which has been concreted until at least 35 days after completion, or longer where weather conditions make it desirable.

REINFORCED CONCRETE APPROACH ROAD TO A FACTORY.

CONCRETE ROAD CONSTRUCTION ON MARSHY GROUND.

By **R. A. B. SMITH, A.C.G.I., A.M.Inst.C.E.**

In cases where concrete roads are to be built on marshy ground, the actual roadway should be of light construction, and it is essential to have the reinforcement at the bottom; reinforcement is also sometimes put near the top to resist contraflection.

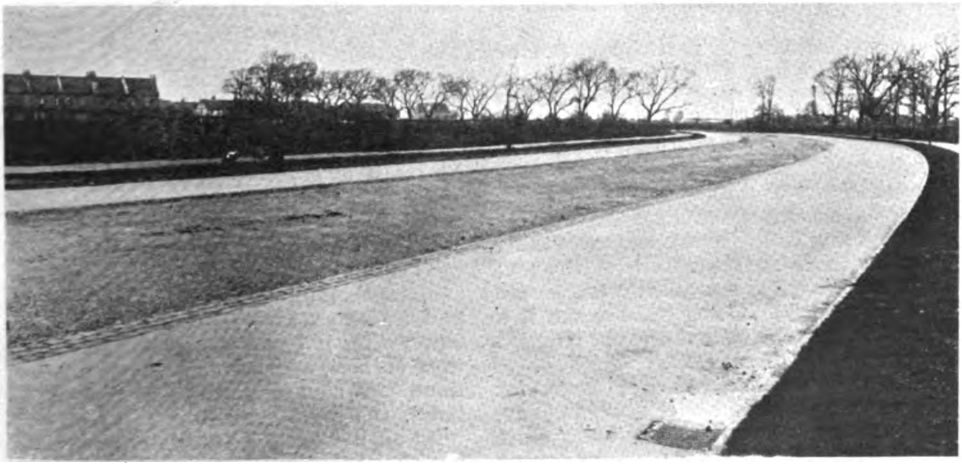
Ditches should be filled in with hard-core or dirty ballast to a depth of about 2 ft. and rolled with a six-ton roller, or a heavier roller if the surrounding surface will stand it. The object is to get the same strength of surface all over; it is not necessary to make the ditch fill stronger than the natural ground. About 3 in. of ashes should be spread all over the hard-core and natural service and rolled in. Rolling in wet weather should be avoided, as the ashes get rolled into the alluvial deposit or mud and wasted. In addition, the ashes make a clean base for the men to work on. The ashes must be thoroughly burned, as it cannot be guaranteed that none of them will get into the concrete during the construction of the road—if unburnt ashes escape into the concrete, "blow holes" will occur.

When concreting, the layer of ashes must be sufficiently damped just in front of the concreting to prevent the cement slurry from soaking through, as this would result in the appearance of "worm holes" in the finished concrete work.

If a tamping screed is used on flat ground, great care must be taken that the side forms are set firm, and particular attention paid that there are pegs close to the joints supporting the forms.

The drains to gulleys across the road should be laid well ahead, and ballast filled on top. It is advisable to place an extra layer of reinforcement at 2 ins. from the top of the concrete extending about 1 ft. or 1 ft. 6 in. over each side of the trench.

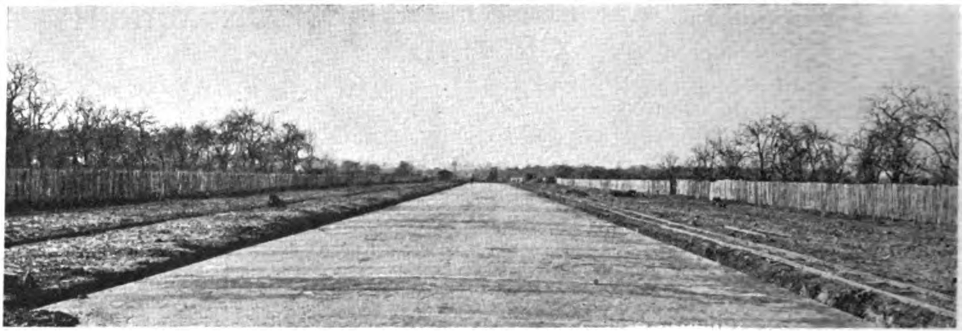
Crossings must be left for future house drains, gas mains, cables, etc. These can be filled in temporarily with a row of granite setts, hard-core, or concrete, which will also act as useful breathers for the road during the initial setting. An extra top reinforcement should be placed in the last 18 in. of a day's work, leaving another 18 in. bare to which the next day's work will knit. The scum pushed to the end of the previous day's work should be cut



[Mr. L. S. McKenzie, City Engineer, Bristol.

REINFORCED CONCRETE ROAD FROM BRISTOL TO AVONMOUTH.

[This road is divided into two portions, each 16 ft. wide and 6 in. thick, by a centre portion reserved for a tramway. The bottom course, 4 in. thick, consists of 1½-in. limestone, ¾-in. limestone chippings, Bideford sand in the proportions of approximately 4½ : 2 : 2½ cu. ft. to 94 lb. of Portland cement; the top 2 in. consists of ⅜-in. to ⅛-in. Penlee chippings, Bideford sand, and Portland cement mixed as follows: 5½ cu. ft. of chippings, 2½ cu. ft. sand, 188 lb. cement.]



[*Mr. A. Dryland, Middlesex County Engineer.*
Mr. J. G. Carey, District Surveyor of Heston and Isleworth.

REINFORCED CONCRETE SECTION OF THE GREAT WEST ROAD.

[This road is built in one course 8 in. thick. The concrete consists of ¾-in. pit ballast, pit sand and Portland cement in the proportions of 4 : 2 : 1. The cost works out at 8*s*. 5½*d*. per sq. yd.]

away to a depth of about 1 in. vertically through a thickness of at least 4 in. before starting the next day's work ; this should be done the next day, even if the work has to be left several days, as it is much more expensive later if the concrete is as good as it should be.

Maximum and minimum temperatures should be taken each day, and it should be remembered that the British Standard tests are generally at about 60 deg. F., whereas the work may be taking place at a temperature of 36 deg. or 80 deg., and that the setting-time is lengthened or shortened according to whether the temperature be low or high.

In hot weather the work must not be stopped for meals. Gangs should be halved through the dinner hour, and if a two-course road is being built the lower and upper courses must be placed on one another as quickly as possible ; the belt, if used, must also follow very close. It is absolutely essential to keep the concrete moist during its initial set, and everything possible should be done to delay the set ; if it is allowed to set quickly cracks are bound to appear. The midday sun is a very tricky problem, and must be guarded against.

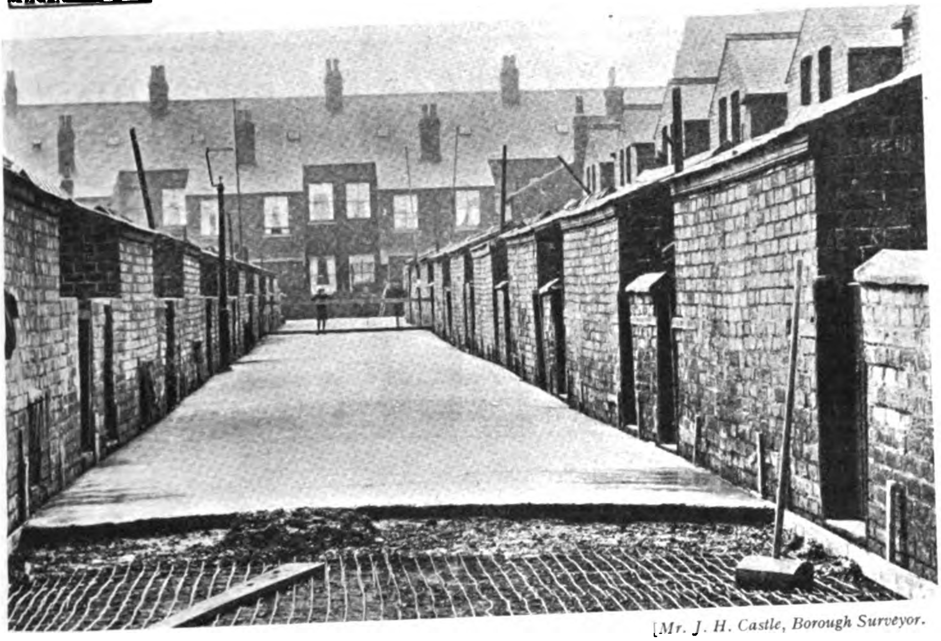
In cool weather half an hour for dinner may be taken by the whole gang, and the top course need not follow quite so closely. A covering of ballast is very desirable, as it retains the moisture from rain and only needs rare watering. In summer it is best to lay "Textilose" (a close-knit canvas) or similar covering direct on the concrete immediately it is free of the belt, but the covering must be kept wet. On removing the covering the road will look patchy, but that appearance is given by the cement slurry, which traffic quickly removes ; granite chippings do not adhere to it, nor is it in any way disturbed by the use of this material. Next day the covering should be removed and the concrete covered with ballast saturated with water.

In a two-course road, both courses should be tamped. The granite chippings must be sharp, very clean, and free from dust. The ballast aggregate for the bottom course must be properly graded and very clean—much cleaner than many so-called "washed" ballasts as supplied. The cement must be very slow setting in summer, but rather quicker in winter in order that the initial set may take place before the night frost appears.

A wooden float should not be used *ad lib.*, as it is liable to undo all the good work already done, unless it is used in a very skilled manner and before the set has taken place.

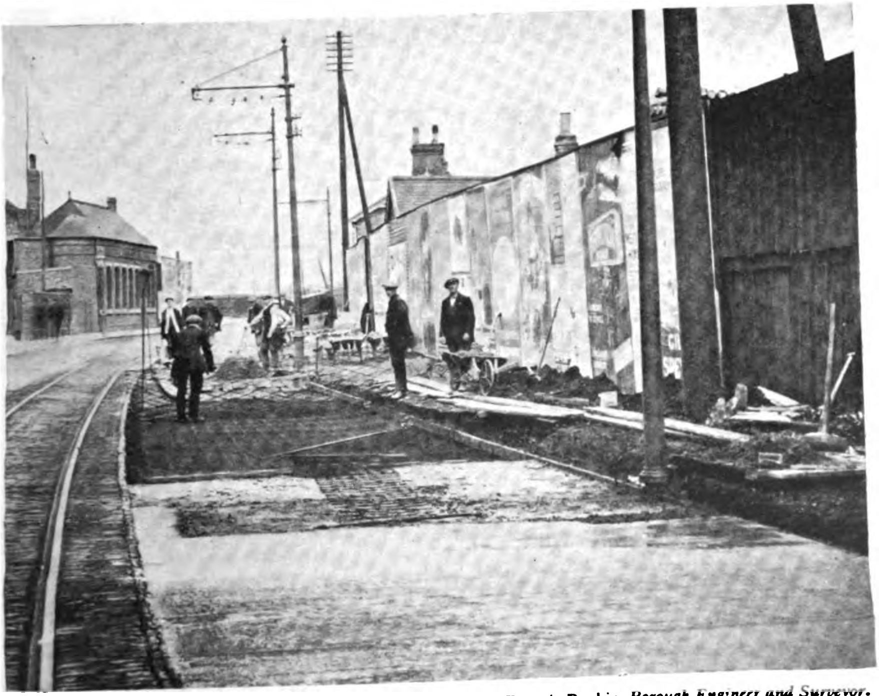
The mix must be a little wetter in summer than in winter, but never sloppy, and never so wet as the men like it. The wetter it is the easier it is to work up a surface, and less tamping has to be done per square yard. On the other hand, the surface must not be tamped until all the cement has been brought to the surface.

In a thin road, i.e., 6 in. thick, rather more cement is necessary than would be allowed for normal reinforced concrete floors, in order that the tamping may leave sufficient cement in the bottom.



[*Mr. J. H. Castle, Borough Surveyor.*

Reinforced Concrete Back Roads (5 in. thick), for the Goole Urban District Council.



[*Mr. Francis Durkin, Borough Engineer and Surveyor.*

Reinforced Concrete Road at West Hartlepool.

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4 cu. ft. to 2 cu. yd.
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CONCRETE CHUTING PLANTS.
Sizes from ½ cu. yd. to 1½ cu. yd.

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STEEL **MAST**
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per hour.

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Concrete Machinery Department,

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SUFFOLK

DOCK HOUSE,
BILLITER STREET,
LONDON, E.C.3

REINFORCEMENT FOR CONCRETE ROADS.

We have to thank Borough Surveyors and the manufacturers of road reinforcement for their kindness in lending us many of the photographs reproduced in this supplement. Practically all the roads are reinforced, and the following list gives the type used and the names of the manufacturers :—

B.R.C. FABRIC.

[Supplied by the British Reinforced Concrete Engineering Co., Ltd., of Iddesleigh House, Westminster, S.W.1]

Tilbury Urban District Council Development and Unemployment Relief Scheme (pp. 262, 263, 268–9, 277–9).

Middlesbrough Corporation Unemployment Relief Schemes (pp. 281–3).

Bristol to Avonmouth Road, for the Bristol Corporation (p. 293).

Reinforced Concrete Section of the Great West Road, Middlesex (p. 294).

Reinforced Concrete Road at Edmonton (p. 264).

Reinforced Concrete Arterial Road at Willesden (p. 289).

Reinforced Concrete Road for the Westhoughton Urban District Council (p. 280).

Reinforced Concrete Back Roads for the Goole Urban District Council (p. 295).

Reinforced Concrete Works Road for the British Tar Products Co., Ltd. (p. 271).

WALKER-WESTON INTERLOCKED DOUBLE-LAYER REINFORCEMENT.

[Supplied by the Walker-Weston Co., Ltd., Empire House, 7 Wormwood Street, London, E.C.3]

Middlesbrough Corporation Unemployment Relief Scheme (pp. 281–3).

Manchester Corporation Arterial Roads (pp. 285–7).

"EXPAMET."

[Supplied by the Expanded Metal Company, Ltd., of York Mansion, Petty France, Westminster, S.W.1]

Reinforced Concrete Approach Road to a Factory (p. 292).

Middlesbrough Corporation Unemployment Relief Scheme (pp. 281–3).

Reinforced Concrete Road at West Hartlepool (p. 295).

"TRIANGLE MESH."

[Supplied by the Indented Bar and Concrete Engineering Co., Ltd., of Queen Anne's Chambers, Westminster, S.W.1]

Reinforced Concrete Road at the Derby Works of Messrs. Rolls-Royce, Ltd. (p. 291).

Reinforced Concrete Goods Yard at East Croydon (London, Brighton and South Coast Railway) (p. 292).

"B AND T" SQUARE MESH.

[Supplied by Messrs. Brown & Tawse, Ltd., of 3 London Wall Buildings, London, E.C.3]

Reinforced Concrete Road (Western Avenue) at Acton (p. 280).

Reinforced Concrete Road at Southampton (p. 287).

Pre-Cast Concrete Road Slabs.

Mr. **Francis Bateman**, of Bateman's Concrete Works, Ltd., writes :—" In your last issue you announce your intention to deal with the all-concrete roads lately laid. May we speak of town and city roads ? Is the present Holborn reconstruction cheap ? Reporters of the Press are told that it will last twenty years with very little repair. We can say as practical concrete manufacturers that Holborn will show potholes, more or less, in less than a month from the time traffic is allowed upon it. The wood blocks or asphalt will sink into the concrete. Why ? Not because the concrete is bad or the work scamped ; on the contrary, Borough Engineers will have, and do have, only the best materials. It is simply that the concrete cannot be given long enough to mature. We have for some time manufactured for inventors different forms of reinforced concrete blocks. They are matured in the works for three months, and when laid are dowelled or locked together. Blocks up to 10 ft. sq. are laid (with the necessary camber). When the line of sewers or other pipes is known the blocks are made smaller, and are lifted out and replaced after the repair to the pipes, etc. When the old road is taken up the pre-cast blocks can be laid as fast as the men clear away, and traffic can follow. If the first cost is more, the saving to the ratepayer must be enormous, directly and indirectly. Also, it must save gas, water, and similar undertakings large sums in maintenance and repairs."

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	6
Clean Shingle, ¾ in. mesh	,,	14	6
,, ,, ½ in. mesh	,,	15	6
Best British Portland Cement	per ton 58*s.* to	63	0
"Ferrocrete" Portland Cement	10*s.* per ton extra.		

BOARDING FOR SHUTTERING—		*Sawn.*		*Wrot.*	
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—			
3 in. by 4 in.	from £22 10*s.*	per	standard.
3 in. by 6 in. and 3 in. by 7 in.	,, £23	,,	,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	14	6
9/16 in. to ½ in. Rounds	,,	14	6
⅜ in. Rounds	,,	15	6
¼ in. Rounds	,,	17	6

MATERIAL AND LABOUR INCLUDING 10 PER CENT. PROFIT.

(*Based on Contracts up to £2,000.*)

PORTLAND CEMENT CONCRETE WELL MIXED WITH CLEAN WATER IN PROPORTION.

1 : 2 : 4—				s.	d.
Do.	do.	in foundation	per yard cube	45	0
Do.	do.	in columns	,, ,,	50	0
Do.	do.	in beams	,, ,,	50	0
Do.	do.	in floor slabs 4 in. thick	per yard super	5	3
Do.	do.	in floor slabs 6 in. thick	,, ,,	7	9
Do.	do.	in floor slabs 9 in. thick	,, ,,	11	3
Do.	do.	in walls 6 in. thick	,, ,,	8	0

(*Add for hoisting* 3*s.* 6*d. per yard cube above ground-floor level.*)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—		s.	d.
From ¼ in. to ⅜ in.	per cwt.	27	6
,, 9/16 in. to ½ in.	,,	25	6
,, ⅝ in. to 2⅞ in.	,,	24	6

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	60	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	50	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11
Do. do. as last in narrow widths	,, ,,	1	1
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1
Do. do. as last in narrow widths	,, ,,	1	3
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

PROSPECTIVE NEW CONCRETE WORK

CARLTON.—*Road and Bridge.*—The West Riding C.C. proposes to construct a new road and bridge over the Aire at Carlton, at a cost of £38,300.

CHELMSFORD.—*Road.*—The Essex C.C. has agreed to contribute 12½ per cent. of the cost of the construction by the Chelmsford Corporation of a new road from Springfield Road to Duke Street. The cost is estimated at £35,707.

CHIPPING SODBURY.—*Roads.*—The R.D.C. has received sanction from the Ministry of Health to borrow £2,658 for a new road from Buckett's Hill to Hatters Lane, and £4,265 in respect of the road from Bucketts Hill to Horseshoe Lane.

DEWSBURY.—*Road.*—The T.C. has decided to proceed with the construction of a new road from Adams Road to Ravenswharf Road.

EALING.—*Bridge.*—The Middlesex C.C. has decided to apply to the Ministry of Health for sanction to borrow £2,918 10s., for the construction of Vicars Bridge.

EDINBURGH.—*Bathing Pond.*—The burgh engineer recommends that the proposed new bathing pond be created by the construction of a concrete wall 100 yards long, and enclosing an area of one half-acre of water. The estimated cost is £11,000.

ELY.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the U.D.C. for sanction to borrow £24,800 for sewerage and sewage disposal works.

HALESOWEN.— *Road.*— The R.D.C. has decided to approve the agreement for the construction of the new road between Birmingham and Wolverhampton.

ILFRACOMBE.—*Pier.*—The U.D.C. is considering a scheme for the reconstruction of a considerable portion of the pier with ferro-concrete.

LIVERPOOL.—*Shed.*—The Mersey Docks and Harbour Board has approved the recommendations of the Works Committee to erect a ferro-concrete three-story shed on the north quay of the Gladstone (No. 2) Dock at a cost of £317,640.

MORECAMBE.—*Road, etc.*—The T.C. has decided to construct a new road to the West end of the town from Bare, at an estimated cost of £75,000. The borough surveyor has been instructed to prepare schemes for a new bathing pool on the foreshore, and the laying out of a park at Bare, estimated to cost £20,000.

PENTRE KENDRICK.—*Bridge.*—The Oswestry R.D.C. has given instructions to the Surveyor to construct a reinforced concrete bridge at Pentre Kendrick, Weston Rhyn, at a cost of £220.

RADCLIFFE.—*Road.*—The U.D.C. has decided to apply to the Ministry of Health for sanction to borrow £11,250 for the construction of a new road between Radcliffe and Bury.

SHREWSBURY.—*Road.*—The T.C. has received sanction from the Ministry of Health to carry out the scheme which provides for the construction of a link road joining the Copthorne Road to the Mount. The total estimated cost is £5,400.

STOKE-ON-TRENT.—*Road.*—The Corporation proposes carrying out an extension of Campbell Road at an estimated cost of £62,500.

SUNBURY.—*Bridge.*—The Middlesex C.C. has decided to apply to the Ministry of Health for sanction to borrow £1,791 for constructing a new bridge over the watersplash, Lower Shepperton Road, Sunbury.

SUNNINGHILL.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the Windsor R.D.C. for sanction to borrow £34,453 and £8,547 for sewerage and sewage disposal works for the parishes of Sunninghill and Sunningdale respectively.

TITCHFIELD.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the R.D.C. for sanction to borrow £6,170 for sewerage and sewage disposal works for the parish.

ULVERSTON. — *Road.* — The R.D.C. has adopted a proposal to construct a new coast road, 60 ft. wide, at an estimated cost of £18,000.

WALTON.—*Road.*—The U.D.C. has received sanction to borrow £12,000 for the construction of a new road from Walton to Frinton.

WEST HARTLEPOOL.—*Jetty.*—A proposal is under consideration for the construction of a new coaling jetty at West Hartlepool.

WIVENHOE.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the U.D.C. for sanction to borrow £15,000 for sewage disposal works.

TRADE NOTICES.

Mr. G. F. Crawford, late of H.M. Forestry Commission, has joined the Roads Department of the British Reinforced Concrete Engineering Co., Ltd., and his address is now No. 47 Whiteladies Road, Bristol.

The Middlesbrough Corporation has placed an order with the Walker-Weston Co., Ltd., Empire House, 7 Wormwood Street, E.C.2, for the reinforcing of four concrete carriage-ways at St. Barnabas' Road, Ayresome Green Road, Roman Road and Eastbourne Road, Middlesbrough. Walker-Weston interlocked double-layer reinforcement has been employed by the Middlesbrough Corporation for seventeen other carriage-ways constructed since 1921, the surfaces of which have worn satisfactorily.

RECENT PATENT APPLICATIONS.

171,978.—R. P. Waagner, Akt. Ges. L. a J. Biro, and A. Kurz : Floor construction.
172,624.—J. V. Meares: Concrete construction.
174,939.—F. De Smacle : Building-blocks.
177,543.—H. Coanda : Floors, walls, roofs, or other structures and reinforced concrete building elements for use therein.
180,337.—J. A. Charpentier : Building blocks.
188,776.—H. D. Fitzpatrick (L. Dusart) : Machine for moulding roofing tiles.
188,779.—R. G. Kirton : Concrete buildings.
188,805.—J. H. P. Wood and R. C. Cameron : Reinforced concrete fencing posts.
188,849.—G. Atterbury : Casting cementitious blocks.
188,990.—J. T. Sandburg : Building block.
188,995.—T. J. J. Albrecht : Building blocks.
189,186.—R. H. Annison : Apparatus for supplying cement grout and concrete in construction works.
189,214.—E. O. Brown : Concrete blocks.
189,290.—R. Thomson : Building blocks.
189,481.—C. Innamorati : Moulding-press for building blocks.
189,519.—E. C. R. Marks (G. E. Rogers) : Building blocks.
189,689.—G. W. Stokes : Reinforcement for concrete.
189,892.—H. Berry : Manufacture of artificial stone and cement.
190,267.—K. Friedrich : Method of producing a cold glaze on concrete, etc.
190,291.—G. F. Dampney : Concrete slab building construction.
190,347.—F. M. Preston : Underwater concrete buckets.
190,405.—N. Johnson : Tamping machines for roads, trenches, etc.
190,849.—A. J. Myles : Concrete.
191,243.—O. Roucka : Process for the manufacture of cement articles and of reinforced-concrete members.
191,561.—H. S. M. Ross : Metal framework and concrete slabs for building construction.
191,840.—C. C. C. Walker : Construction of hollow or solid walls of concrete, etc.
191,908.—W. L. Scott : Construction of hollow reinforced concrete flooring.
192,124.—D. H. Stent : Reinforced concrete railway sleepers.
192,198.—H. Wakelam : Reinforcements for concrete.
192,260.—F. B. Dehn (Blaw-Knox Co.) : Wall form or mould.
192,279.—W. W. Gibbings : Concrete floors.
192,294.—A. J. Rowley : Walls and floors.
192,547.—C. J. Upton : Double or hollow concrete walls.
192,859.—L. Parkin : Building-blocks.
193,269.—A. W. Jeffery : Metal reinforcement of concrete constructions.

TENDERS ACCEPTED.

ASHBOURNE.—*Reservoir.*—The U.D.C. has, provisionally, accepted the tender of Messrs. J. K. Ford & Weston, Ltd., Omaston Road, Derby, at £1,944, for the construction of a reinforced concrete service reservoir on the Trussed Concrete Steel Co., Ltd., system.

CREWE.—*Bridge.*—The Corporation has accepted the tender of Messrs. S. & J. Smethurst, Oldham, at £3,845 17*s.* 11*d.* for the construction of the new Alton Street bridge.

HAVERFORDWEST.—*Concrete Tubes.*—The T.C. has accepted the tender of Messrs. J. Ellis & Sons, Ltd., Leicester, for the supply of standard concrete tubes in connection with the High Street sewer.

ENFIELD.—*Sewer.*—The U.D.C. has accepted the tender of Messrs. G. Bell & Sons, Ltd., Tottenham, for the construction of a 24-in. concrete tube storm-water sewer with manholes, and the partial concreting of an iron shield pipe under the New River.

STEPNEY.—***Foundation Beds.***—The Borough Council has accepted the tender of the Hydraulic Mining Cartridge Co., Silverwell Street, Bolton, at £293 15*s.*, for concrete foundation beds at the Limehouse generating station.

NEW COMPANIES REGISTERED.

CHEAM CO-PARTNERS, LTD. (187407.) Registered January 29. 5, Wine Office Court, E.C. Builders and contractors, concrete and other engineers, etc. Nominal capital, £1,000 in 975 8 per cent. Par Pref. Shares of £1 each, and 500 Ordinary Shares of 1*s.* each. Directors : T. N. Blackhouse, 23 Craigen Avenue, Addiscombe ; W. H. Thompson, 131 Algernon Road, Lewisham, S.E. Qualification of Directors : 1 Share. Remuneration of Directors : To be voted by Company.

CONCRETE UNIT CO., LTD. (188009). Registered February 22. Ingersoll Road, Ashburton Road, Trafford Park. Concrete manufacturers. Nominal capital, £2,500 in 2,500 Shares of £1 each. Directors : H. E. Holden, 4 Willow Tree Road, Altrincham (Chairman and Managing Director) ; E. Haworth, Hazeldene, Marple Bridge, near Stockport. Qualification of Directors : £100.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

MAY 1923 VOL. XVIII. No. 5

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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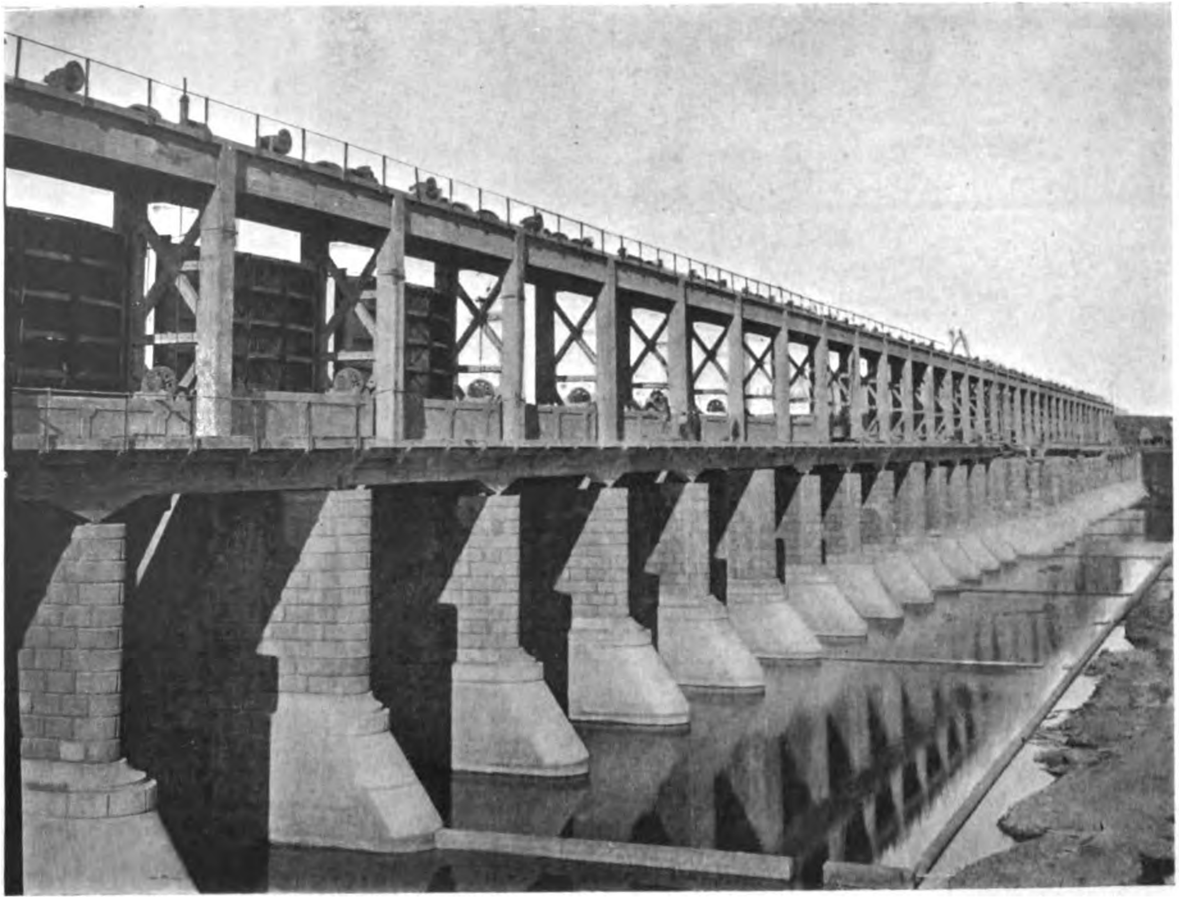

The Vaal River Barrage: General View. (*See* p. 321.)

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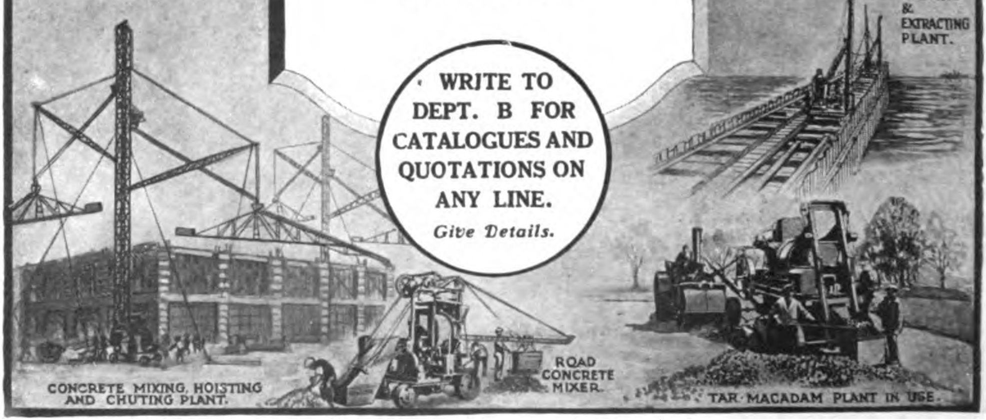
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ROAD CONCRETE MIXER.
TAR-MACADAM PLANT IN USE.

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume XVIII. No. 5. LONDON, MAY, 1923.

EDITORIAL NOTES.

STANDARDS OF CONSTRUCTION OF SMALL HOUSES.

FROM a report of the Building Code Committee of the U.S. Department of Commerce, entitled " Recommended Minimum Requirements for Small Dwelling Construction," it appears that the non-uniformity of building codes, their irksomeness and sometimes absurdity, are not confined to this country, but have been a factor in hampering the efforts to make up the arrears of dwelling-houses in the United States. The Committee was set up not only to consider a uniform building code throughout the country for this type of house, but also to prepare " a standard practice based not on inherited practice, but on the real need for strength in small buildings. . . . The methods and practices now in use in the design and construction of dwelling-houses are the result of blindly following tradition and past custom. There seems to be no other reason for the thickness of masonry walls used for ordinary one-story and two-story dwellings than a survival of the belief that such walls should act to a large extent as parts of a fort. . . . The actual loads coming on the walls of the ordinary one-story or two-story house are ridiculously small."

The report of the Committee, if adopted, would remove many of the limitations which the common practice in house construction in America seemed to require. For instance, instead of 12-in. brick walls for two-story buildings, a thickness of 8 in. is permitted. Recognition is also given to other types of construction, and the use of concrete, both plain and reinforced, is provided for by allowing exterior walls of this material to be 6 in. thick. Walls of hollow concrete block 8 in. thick are allowed for the uppermost 20 ft. of buildings limited to 30 ft. high. Double walls of reinforced concrete cast *in situ*, providing an insulation air space, may be constructed with a total thickness for the two walls of 6 in. The use of large pre-cast units each of which may be one-story high and form an entire one-story wall of one side of the house is permitted.

The adoption of this report should do much to cheapen and expedite small house construction in the United States. In its endeavour to get houses built the Ministry of Health has during the past few years relaxed many of the antiquated local by-laws, and freed the building industry from many of the shackles which for so long hampered the erection of small houses, but a thorough investigation here on the same lines as those covered in the report referred to would undoubtedly disclose many sources of economy in house building which would materially help in the production of the urgently-needed houses, and at the same time assist private enterprise to return to its legitimate occupation.

WAR AND BUILDINGS.

AMID all the difficulties which are its aftermath no one likes to be unnecessarily reminded of the war, but at the same time we cannot afford to overlook any small crumbs of knowledge which can be garnered from that welter of waste. We therefore welcome the publication by the British Fire Prevention Committee of No. 40 in its well-known series of " Red Books," entitled " Air Raid Damage in London " (London : 8 Waterloo Place, S.W.1 ; price 5*s*. net), prepared by Major E. C. P. Monson and Mr. Ellis Marsland, which contains an illustrated record of the effect of aircraft attack on certain public and private buildings in London. The photographs of wrecked buildings clearly show that in nearly all the cases under review the damage was caused not so much by the actual explosion of the bombs as by the fires which followed, and that the buildings of fire-resisting construction suffered much less than others. This is what one would naturally expect owing to the greater strength which accompanies an increased degree of fire-resistance. The introduction to the book states that from the structural point of view comparative immunity from the danger of explosive bombs can apparently be obtained in substantial modern buildings having concrete floors, and almost entire immunity if the inmates of the buildings have three concrete floors above them. From the investigations it became clear that a series of concrete floors affords the best protection against the average explosive bomb and almost entire protection on any floor against incendiary bombs. It is recommended that in view of future possible air-raids all buildings should be built with concrete floors and roofs, but in spite of Washington's dictum that " To be prepared for war is one of the most effectual means of preserving peace," it seems almost too much to anticipate that this will be carried out solely as an insurance against attack by air, especially as we do not know whether bombs dropped from the air will be the favourite means of exterminating the enemy civilian population when the next war comes along. The book might have done more to attain its objects as regards commercial buildings if it had pointed out that in the majority of cases concrete and reinforced concrete are the cheapest as well as the safest materials for the construction of floors and roofs, but it serves a very useful purpose in indicating the best methods for obtaining shock and fire-resisting construction which should be valuable to the Government in designing forts and similar buildings.

LONDON'S ROADS AND STREETS.

AN excellent idea of the enormous cost of keeping in repair the streets of London can be obtained from the sixteenth annual report of the Metropolitan Committee on Materials and Means of Paving the Streets of London, which contains a mass of information tabulated in a comprehensive manner on the repair works carried out by the various local authorities during the past year. The total sum thus expended during the year is not given, but when it is considered that the eternal " Road-up " notice means that anything up to £3 or more per yard super is being spent over those areas it can be imagined that the total for the whole of London must be an almost staggering figure. The report outlines Mr. Arthur Harrison's experiments with rubber paving at Southwark, but makes no mention of the all-concrete roads in the same Borough. The report as a whole, however, appears to be one of the strongest arguments in favour of the all-concrete road that could possibly be put forward. An analysis of the report shows that all the roads included, except those carrying a very limited amount of traffic, have a concrete or reinforced concrete foundation, varying in depth from 6 in. to 14 in., covered with wood blocks, asphalt, granite setts, or bituminous paving, and that the cost of laying the wood blocks or granite setts in all cases is considerably more than the cost of laying the concrete bed. Lillie Road, Fulham, for instance, cost 15*s*. 5*d* per yard for a 9 in. to 10 in. concrete base, and 22*s*. 10*d*. per yd. for deal blocks; Mare Street, Hackney, cost 15*s*. 2*d*. per yd. for a 12-in. concrete foundation, and 21*s*. 7*d*. per yd. for soft-wood blocks; Lambeth Road cost 18*s*. 1*d*. per yd. for a 9-in. concrete foundation, and 31*s*. 6*d*. per yd. for deal blocks; part of the Albert Embankment cost 17*s*. 3*d*. per yd. for a 9-in. concrete bed, and 31*s*. 5*d*. per yd. for granite setts. In some of these instances, of course, the foundations were not completely renewed, and the cost of the surfacing includes maintenance for periods up to five years, but nevertheless the figures show what enormous savings could be effected if the necessary foundation were laid so that its surface would be hard and dense enough to withstand the wear and tear of traffic. The roads illustrated in our last issue and the statements of Borough Surveyors who have tried this type of road show that with careful supervision and good materials bare concrete will resist the disintegrating effects of modern traffic as well as any expensive surfacing material with which it is the present practice to cover it, and that an opportunity is thus presented practically to halve the millions spent annually in London on new roads and road repair.

THE WEMBLEY STADIUM.

It is not often a new structure is subjected to such a severe test the first time it is used as was the new Stadium at the British Empire Exhibition at Wembley on the occasion of the Cup Final on April 28. That it has successfully withstood the surging of a very much larger crowd than it was ever anticipated it would be called upon to carry is an excellent testimonial to the strength of a reinforced concrete structure. The authorities are to be congratulated on the selection for the construction of the Stadium of a material which renders almost impossible such a calamity as befell the stand at Glasgow which, on a similar occasion, collapsed with loss of life in 1902.

MOMENTS IN CONTINUOUS BEAMS FOR VARYING RATIOS OF LIVE LOAD TO DEAD LOAD.

By CHARLES S. GRAY, B.Sc.Eng., A.M.I.Struct.E.

WHEN designing continuous beams in reinforced concrete the ratio of live load to dead load varies with every structure, and often indeed varies with every beam unless standard units and standard spacing of beams are adopted. This variation makes it often difficult to decide upon the bending moments to be resisted, and it is the purpose of this article to present in a convenient form figures giving the maximum moments in the several spans for the varying ratios of live load to dead load.

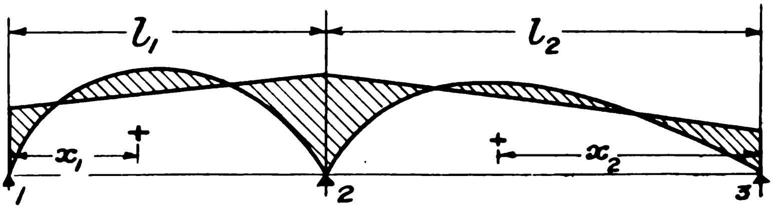


FIG. 1.

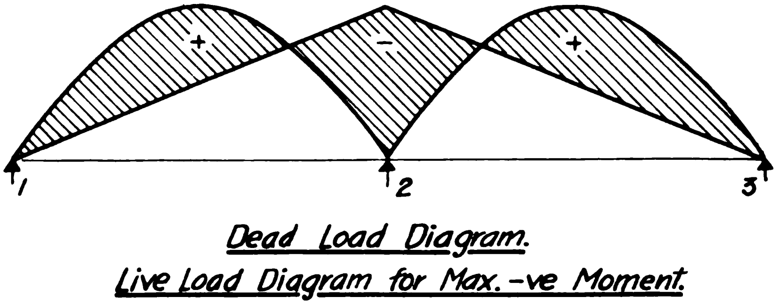


FIG. 2.

For the present we will consider only beams uniformly loaded having equal spans, and having also a constant moment of inertia. In practice the live load is usually from one to six times the dead load, and figures are given to cover this range for beams having from two to six spans. We shall not treat spans partially loaded with live load, but spans either fully loaded or loaded with dead load only.

TWO-SPAN BEAMS.

We will consider first a simple case, that of a two-span beam having a distributed dead load of W_d per span and a distributed live load of W_l per span, the spans being equal.

Then we have by Clapeyron's Theorem of Three Moments for any continuous beam,

$$B_1 l_1 + 2B_2(l_1 + l_2) + B_3 l_2 = 6\left\{\frac{A_1 x_1}{l_1} + \frac{A_2 x_2}{l_2}\right\},$$

wherein—

l_1, l_2 = lengths of spans,
B_1, B_2, B_3 = support bending moments,
A_1, A_2 = areas of respective free bending moment diagrams,
x_1, x_2 = distances of centres of gravity of free bending moment diagrams from supports 1 and 3. (*See Fig.* 1.)

Then, considering dead load on our two-span beam, we have :

Maximum height of free B.M. diagram

$$= \frac{W_d l}{8} = \cdot125 W_d l.$$

$$\text{Then } Ax \text{ for one span } = \frac{W_d l}{8} \cdot \frac{l}{2} \cdot \frac{2l}{3}$$

$$= \frac{W_d l^3}{24}.$$

Then, applying Clapeyron's Theorem :

$$4B_2 = \frac{2 \times 6}{l^2}\left(\frac{W_d l^3}{24}\right)$$

$$\therefore \; B_2 = \cdot 125 W_d l \quad . \quad . \quad . \quad . \quad \text{(i)}$$

The same reasoning applies when both spans are fully loaded with live load and the bending moment due to that load $= \cdot 125 W_l l$. (*See Fig.* 2) . . . . (ii)

We now require to know the maximum positive bending moment near the centre of the span : this is found as follows :—

Let $W_d = p_d l$, then the negative bending moment due to the support bending moment

$$= \frac{p_d l x}{8},$$

and the positive bending moment for a freely supported beam

$$= \left(\frac{p_d l x}{2} - \frac{p_d x^2}{2}\right),$$

where x is the distance between the end support and the section under consideration. (*See* Andrew's *Theory of Structures*.)

Then the total bending moment at that point is given by

$$\frac{p_d l x}{8} - \frac{p_d l x}{2} + \frac{p_d x^2}{2}.$$

This is a maximum when

$$\frac{d}{dx}\left(\frac{lx}{8} - \frac{lx}{2} + \frac{x^2}{2}\right) = 0 :$$

$$\text{i.e. when} \quad \frac{l}{8} - \frac{l}{2} + x = 0 :$$

$$\text{i.e. when} \quad x = \frac{3l}{8},$$

where x is measured from the outer support.

Then maximum positive B.M.

$$= \frac{p_d l}{2} \cdot \frac{3l}{8} - \frac{p_d}{2}\left(\frac{3l}{8}\right)^2 - \frac{p_d l}{8} \cdot \frac{3l}{8}$$

$$= p_d l^2\left(\frac{3}{16} - \frac{9}{128} - \frac{3}{64}\right)$$

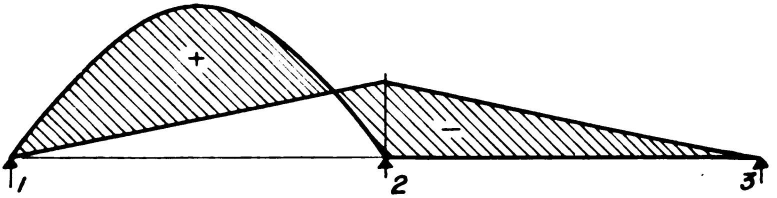
$$= \frac{9}{128} p_d l^2$$

$$= \cdot 0703 W_d l \quad . \quad . \quad . \quad . \quad . \quad \text{(iii)}$$

Considering live load, the maximum positive bending moment will be given when one span only is loaded, as in *Fig.* 3. Then, using Clapeyron's Theorem as before,

$$4B_2 = \frac{6}{l^2}\left(\frac{W_l l^3}{24}\right)$$

$$\therefore \; B_2 = \cdot 0625 W_l l \quad . \quad . \quad . \quad . \quad \text{(iv)}$$



Live Load Diagram for Max. +ve Moment.

Fig. 3.

Then if

$$W_l = p_l \times l,$$

$$B_2 = \frac{p_l l^2}{16}.$$

The equation to the free bending moment diagram is the same as before, and so the total bending moment at any point x is given by

$$\frac{p_l l x}{16} - \frac{p_l l x}{2} + \frac{p_l x^2}{2}.$$

This is a maximum when

$$\frac{l}{16} - \frac{l}{2} + x = 0 ;$$

$$\text{i.e., when } x = \frac{7l}{16}.$$

Then maximum positive B.M.

$$= -\frac{7}{256} p_l l^2 + \frac{7}{32} p_l l^2 - \frac{49}{512} p_l l^2,$$

$$= p_l l^2\left(-\frac{7}{256} + \frac{7}{32} - \frac{49}{512}\right),$$

$$= \frac{49}{512} p_l l^2,$$

$$= \cdot 0957 W_l l \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(v)}$$

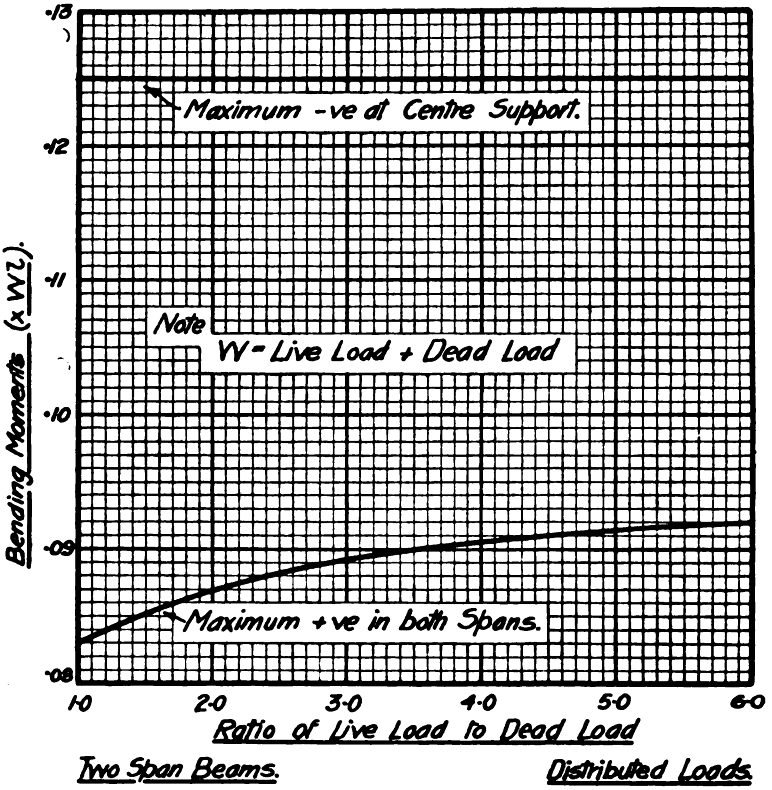


Fig. 4.

Two-Span Beams
Maximum Bending Moments (× Wl).

Ratio of Live Load : Dead Load.	+ Near Centre of Span	— At Support.
1 : 1	$\frac{\cdot 0703 + \cdot 0957}{2} = \cdot 083$	·125
2 : 1	$\frac{\cdot 0703 + 2 \times \cdot 0957}{3} = \cdot 087$	·125
3 : 1	$\frac{\cdot 0703 + 3 \times \cdot 0957}{4} = \cdot 089$	·125
4 : 1	$\frac{\cdot 0703 + 4 \times \cdot 0957}{5} = \cdot 0906$	·125
5 : 1	$\frac{\cdot 0703 + 5 \times \cdot 0957}{6} = \cdot 0915$	·125
6 : 1	$\frac{\cdot 0703 + 6 \times \cdot 0957}{7} = \cdot 092$	·125

Table 1.

The maximum negative bending moment due to live load will be given when both spans are loaded and is equal to ·125$W_l l$ as stated before.

Now let $W = W_d + W_l$, then the actual maximum B.M.'s on the beam caused by the combination of live and dead loads can be found as tabulated at side.

These values are plotted and give the curve shown in *Fig.* 4, from which the moments applicable to any given range of dead load and live load between the ratios of 1 : 1 and 1 : 6 can be read off.

THREE-SPAN BEAMS.

Working on the lines of the previous case we find the values for the maximum moments in the beam given in Table 2.

Maximum Moments, Three-Span Beams.

	+ Near Centre of End Span.	— At Second Support.	+ Near Centre of Mid Span.
Dead Load .	·08$W_d l$	·1$W_d l$	·025$W_d l$
Live Load .	·1$W_l l$	·117$W_l l$	·075$W_l l$

Table 2.

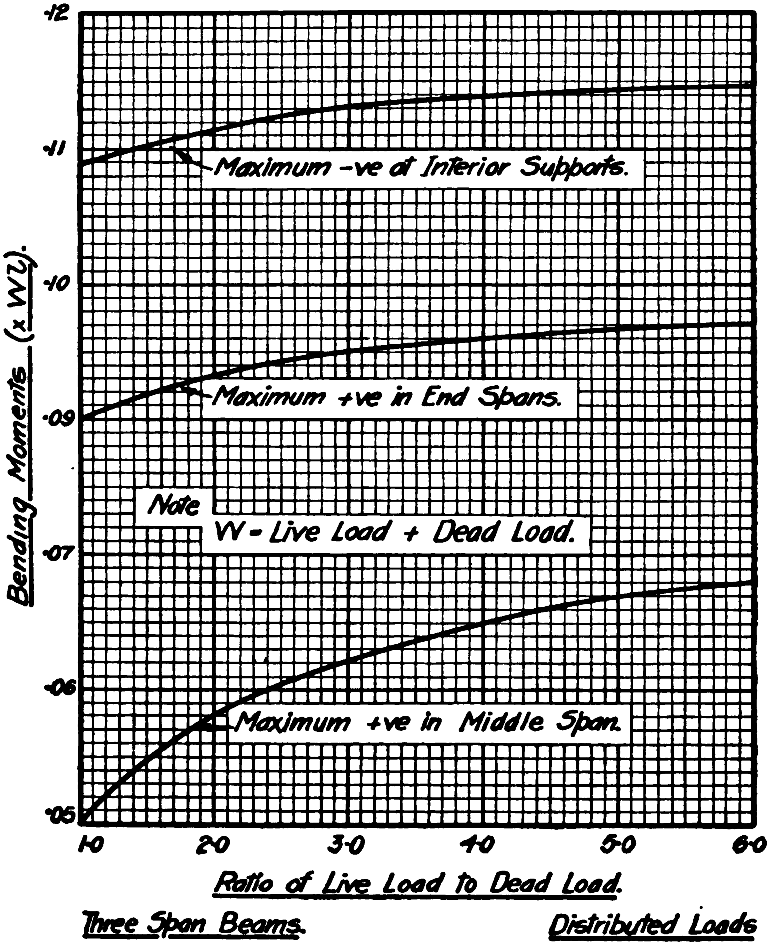


FIG. 5.

These values are then combined in the same manner as in Table 1, and we get Table 3, giving the maximum moments for the varying ratios of dead load to live load, and these values are plotted in *Fig.* 5.

THREE SPAN-BEAMS.

MAXIMUM BENDING MOMENTS (× *Wl*)

Ratio of Live Load : Dead Load.	+ Near Centre of End Span.	− At Second Support.	+ Near Centre of Mid Span.
1 : 1	·090	·109	·050
2 : 1	·093	·111	·058
3 : 1	·095	·113	·062
4 : 1	·096	·114	·065
5 : 1	·097	·114	·067
6 : 1	·097	·115	·068

TABLE 3.

FOUR-SPAN BEAMS.

For four-span beams there are fifteen different arrangements of loading with live load, but in general it may be said that the maximum positive moments at the centres of the spans due to live load in a beam of several spans are given when alternate spans only are loaded. Maximum negative moments at interior supports are given when two adjoining spans are loaded and then each alternate span (Messrs. Turneare and Maurer*).

We then get from the Three Moments

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* *Principles of Reinforced Concrete Construction*, Turneare and Maurer.

Theorem the maximum values for the live load and dead load moments given in Table 4.

MAXIMUM MOMENTS, FOUR-SPAN BEAMS.

	End Spans.		Interior Spans.	
	+ Near Centre.	— At Second Support.	+ Near Centre.	— At Support.
Dead Load	·071 *Wdl*	·107 *Wdl*	·036 *Wdl*	·071 *Wdl*
Live Load	·098 *Wll*	·120 *Wll*	·081 *Wll*	·107 *Wll*

TABLE 4.

FOUR-SPAN BEAMS.
MAXIMUM BENDING MOMENTS (× *Wl*)

Ratio of Live Load : Dead Load.	End Spans.		Interior Spans.	
	+ Near Centre.	— At Second Support.	+ Near Centre.	— At Support.
1 : 1	·085	·114	·059	·089
2 : 1	·089	·116	·066	·095
3 : 1	·091	·117	·070	·098
4 : 1	·092	·117	·072	·100
5 : 1	·093	·118	·0735	·101
6 : 1	·094	·118	·075	·102

TABLE 5.

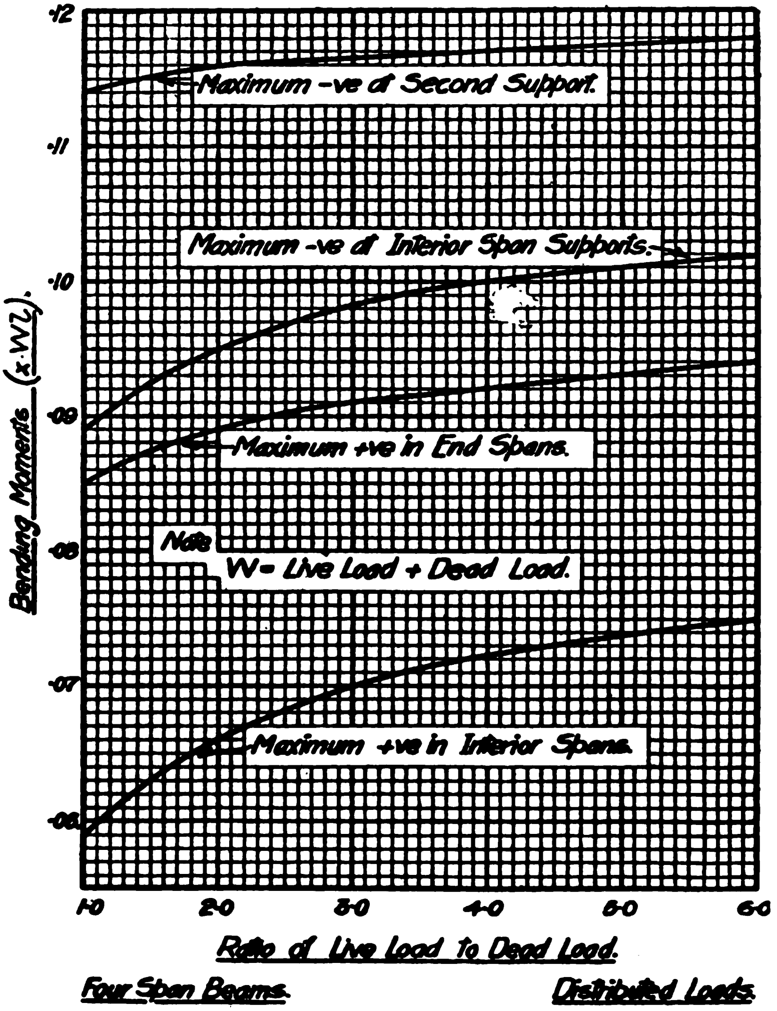

FIG. 6.

In the case of the interior spans the

maximum value given is that attained in any one interior span.

By combining these figures we obtain the values given in Table 5, from which the curves in *Fig.* 6 are derived.

FIVE-SPAN BEAMS.

Proceeding along the lines previously indicated, we can obtain the following values for maximum live load and dead load moments in five-span beams, but the figures below are extracted from those given by Messrs. Turneare and Maurer* and Geen.†

MAXIMUM MOMENTS, FIVE-SPAN BEAMS.

	End Spans.		Interior Spans.	
	+ Near Centre.	− At Second Support.	+ Near Centre.	− At Support.
Dead Load	·072Wdl	·105Wdl	·046Wdl	·079Wdl
Live Load.	·099Wll	·120Wll	·086Wll	·111Wll

TABLE 6.

The values given for interior spans are the maxima attained in any one of the several spans.

From these figures Table 7 is obtained by the method used for Table 1, and Table 7 is set out graphically in *Fig.* 7.

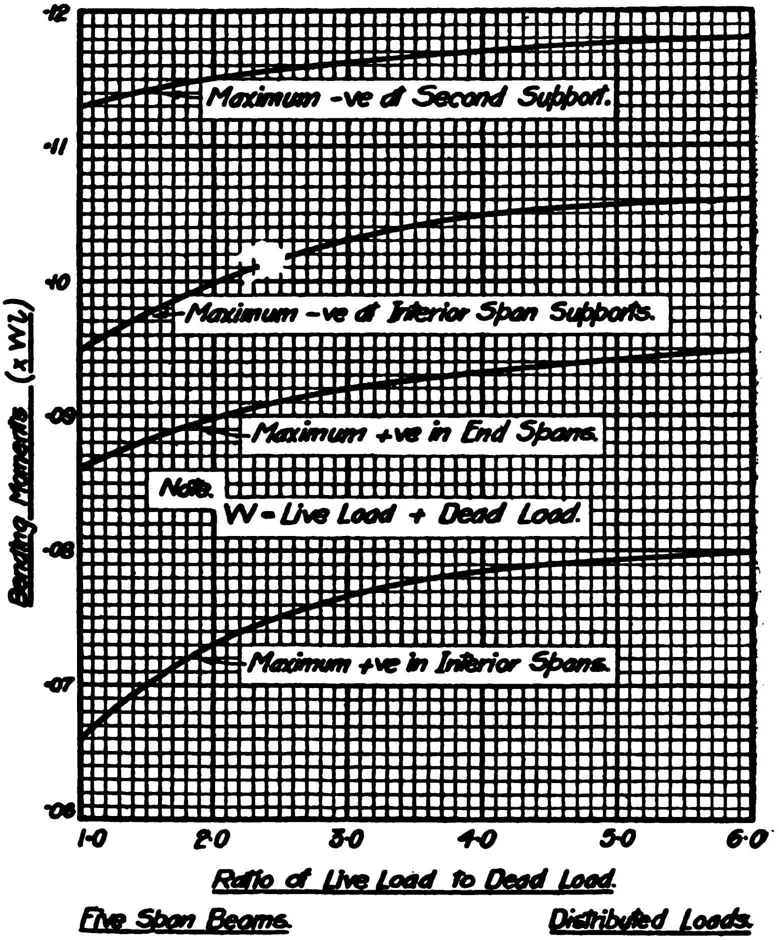

FIG. 7.

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* *Principles of Reinforced Concrete Construction,* Turneare and Maurer.

† *Continuous Beams in Reinforced Concrete,* Geen.

FIVE-SPAN BEAMS.
MAXIMUM BENDING MOMENTS (× Wl)

Ratio of Live Load : Dead Load.	End Spans.		Interior Spans.	
	+ Near Centre.	− At Second Support.	+ Near Centre.	− At Support.
1 : 1	·086	·113	·066	·095
2 : 1	·090	·115	·073	·100
3 : 1	·092	·116	·0765	·103
4 : 1	·093	·117	·078	·105
5 : 1	·094	·1175	·079	·1055
6 : 1	·095	·118	·080	·106

TABLE 7.

SIX-SPAN BEAMS.

The figures for six-span beams are obtained in a similar manner to those for five-span beams, and are tabulated in Tables 8 and 9 and shown graphically by *Fig.* 8.

MAXIMUM MOMENTS, SIX-SPAN BEAMS.

	End Spans.		Interior Spans.	
	+ Near Centre.	− At Second Support.	+ Near Centre.	− At Support.
Dead Load	·072$W_d l$	·106$W_d l$	·043$W_d l$	·086$W_d l$
Live Load	·099$W_l l$	·120$W_l l$	·084$W_l l$	·116$W_l l$

TABLE 8.

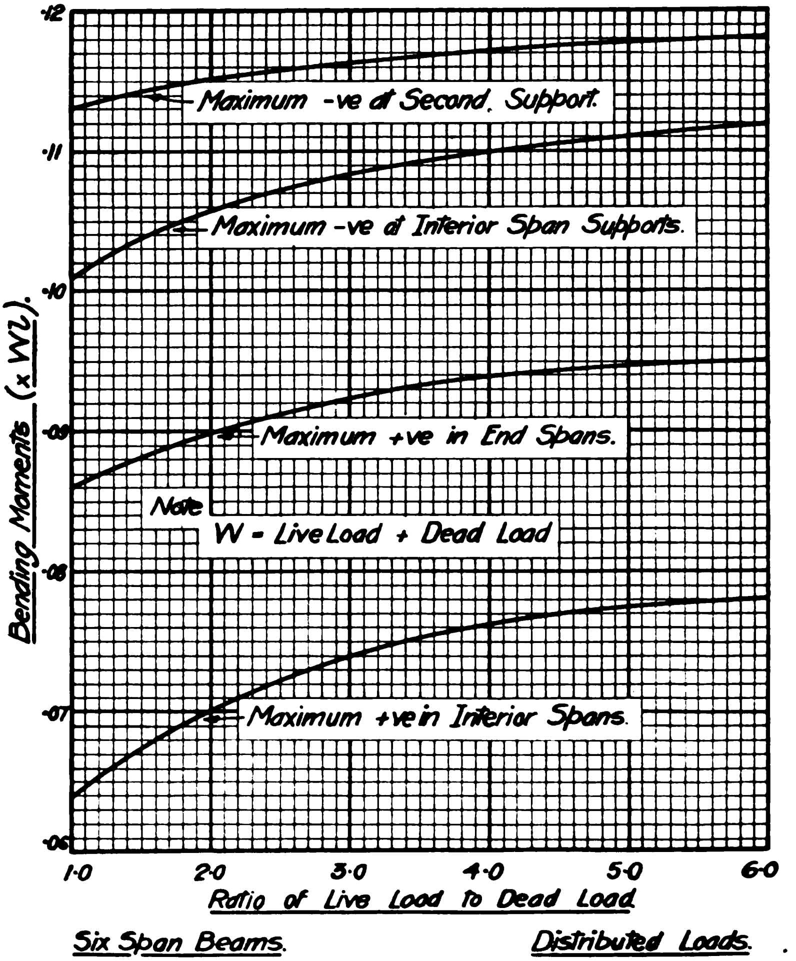


FIG. 8.

SIX-SPAN BEAMS.

MAXIMUM BENDING MOMENTS (× Wl).

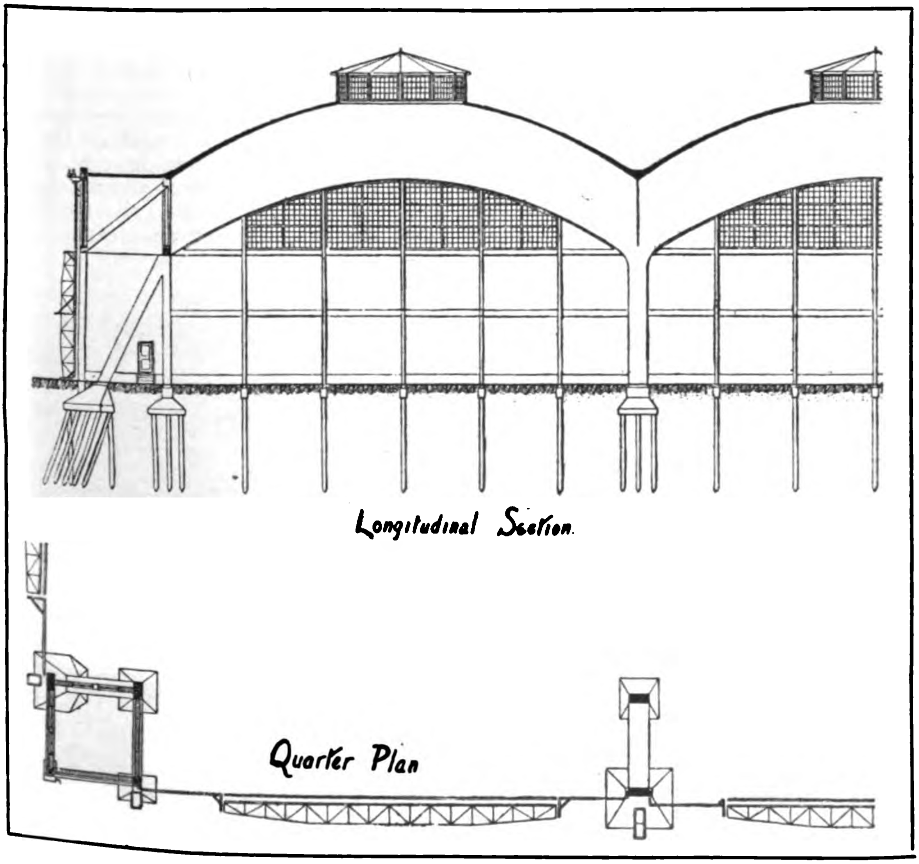
Ratio of Live Load : Dead Load.	End Spans.		Interior Spans.	
	+ Near Centre.	− At Second Support.	+ Near Centre.	− At Support.
1 : 1	·086	·113	·064	·101
2 : 1	·090	·115	·070	·106
3 : 1	·092	·116	·074	·108
4 : 1	·094	·117	·076	·110
5 : 1	·0945	·118	·077	·111
6 : 1	·095	·118	·078	·112

TABLE 9.

HANGAR FOR HYDRO-AEROPLANES AT REVAL.

THE most interesting feature of the newly-built hangar at Reval is its unusual shape, which is one for which reinforced concrete is peculiarly suitable. The building is 412 ft. long and 170 ft. wide, so as to contain three aeroplanes, each in a separate compartment. The dome-shaped roofs are supported on vertical and oblique pillars, the latter carrying the outward thrust of the domes. The only part of the structure which is not built of reinforced concrete is the lantern in each dome, which is of wood, for the sake of lightness.

The roof is only 6 in. in average thickness, some parts being only 3 in. thick, and the greatest thickness 28 in. Ample bays for stores and repair shops are provided. The main doors are built in a vertical concrete framework, which projects from the main structure and is supported on eight pillars. The ends opposite the doors are permanently closed.—*Le Constructeur de Ciment Arme.*



HANGAR FOR HYDRO-AEROPLANES AT REVAL.

AMERICAN METHODS IN CONCRETE WORK.

By ALBERT LAKEMAN, M.S.A.

(*Concluded from p.* 223).

TRANSPORT OF CONCRETE.

In American work the transport of the mixed concrete from a central mixing plant to the various parts of the site is a question which receives considerable attention, because the importance of keeping the whole organisation working at a uniform speed is realised. When the work covers a large area the usual practice is to install one or more mixers at fixed points where the whole of the materials are dealt with, and from these points a good transport system is arranged to cope with the maximum output of the mixers. This method has distinct advantages, which include:

(*a*) All the raw materials can be brought up to the mixer and unloaded direct from the railway wagons with stationary unloading equipment operated by a single power unit which also operates the mixer;

(*b*) A large-capacity batch mixer can be adopted which will give the maximum output with the minimum labour;

(*c*) The gauging and mixing of the concrete can be easily controlled;

(*d*) There is no loss of time through moving the concrete mixing plant.

The writer did not see any example of the use of a continuous type of concrete mixer in America. The central mixing plant is arranged to give an output equal to the maximum that can be obtained with the capacity of the mixer installed, and this capacity is, of course, based on the total amount of concrete to be placed in a certain period. The transport of the mixed concrete is the next link in the sequence of operations, and care is exercised to arrange this in a manner which will enable the full output from the mixer to be taken without delays or slowing down of operations. Narrow-gauge track is frequently laid down for the running of concreting trains with special shaped cars and small steam or electric locomotives. The tracks and locomotives are also useful for the transportation of reinforcement, forms, and any materials used in the scheme, and loop lines, branches, and sidings are provided which allow trains to pass each other in order to give a continuous flow to and from the essential points.

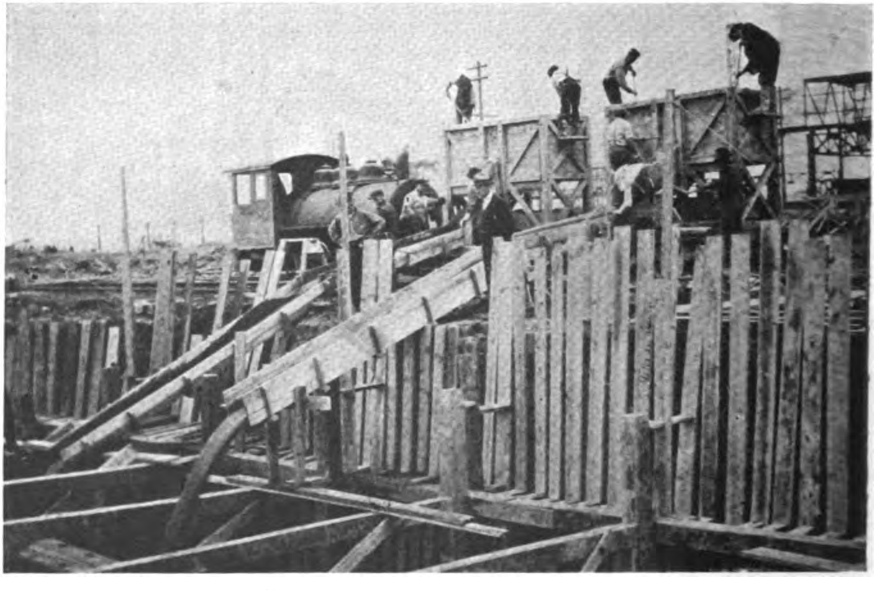


Fig. 16.—Train for Transporting Concrete.

A typical concreting train discharging its contents is shown in *Fig.* 16, where an underground pipe subway is in process of construction. In this case two special cars are in use on one train, these cars having a sloping bottom leading to a discharge hopper on one side to enable the mixed concrete to be discharged direct into wooden chutes, as shown. It will be noticed that the cars are raised well above the axles of the wheels, and this is necessary to allow the discharge-hopper to be kept sufficiently high to give a good inclination to the chutes. Small platforms are provided at the ends of each car for the use of workmen when clearing the sides and bottom of concrete at each discharge. Labourers are employed with rakes and shovels to draw the concrete down the chutes when the inclination is small, and to assist generally in the flow of the concrete. When the train is emptied and it has left on the return journey to the mixer these labourers proceed with the tamping and levelling off in the forms. The example illustrated is important because the subway under construction was over 3,500 ft. long, 10 ft. wide, and 10 ft. 6 in. high, and under a portion of this, for a length of 1,600 ft., a storm-culvert was also formed having a capacity of over 6,000,000 gallons per hour. The entire work was executed in reinforced concrete, and two trains were employed for the transport of the mixed concrete to the point of deposit. One train would be loading at the mixing-plant while the other was discharging, and by means of whistle signals they would pass on a loop line midway between the two points. At the mixing plant the concrete was elevated and discharged by gravity into a receiving-hopper adjacent to the tracks, and this hopper was raised above the level of the top of the cars to permit direct loading from the former to the latter. Each train transported 2 cu. yds., and the average time allowed for loading, outward journey, discharge, and return journey was about 5 minutes, which meant that 48 cu. yds. of concrete were being placed per hour with the two trains. The system worked admirably, and the men engaged on the various operations were kept fully employed during the day ; the work was, in consequence, executed at a reasonable cost. The transport of mixed concrete in wheel-barrows so often adopted in this country is tedious and costly, and it is never adopted on any work of importance in America. When it is necessary to transport by hand over short distances between the receiving point

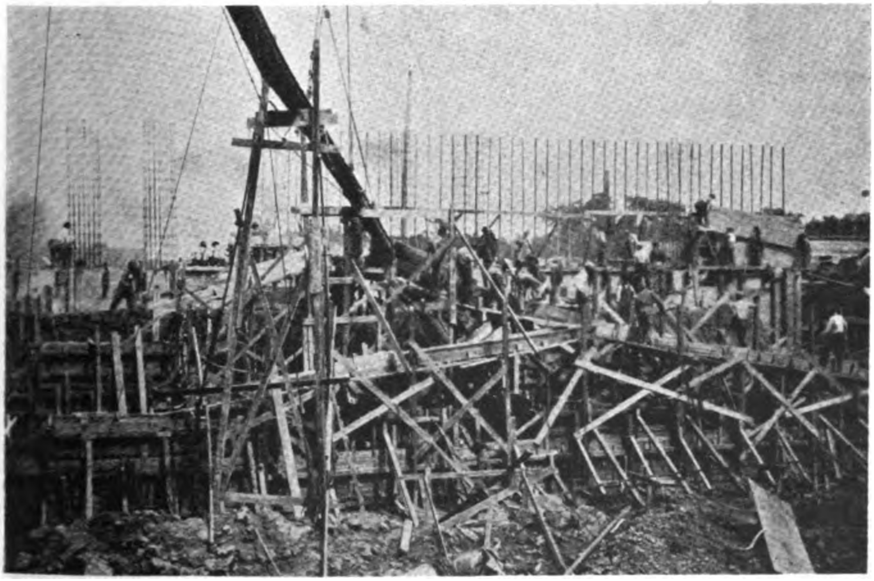


Fig. 17.—Example of Chutes in American Construction.

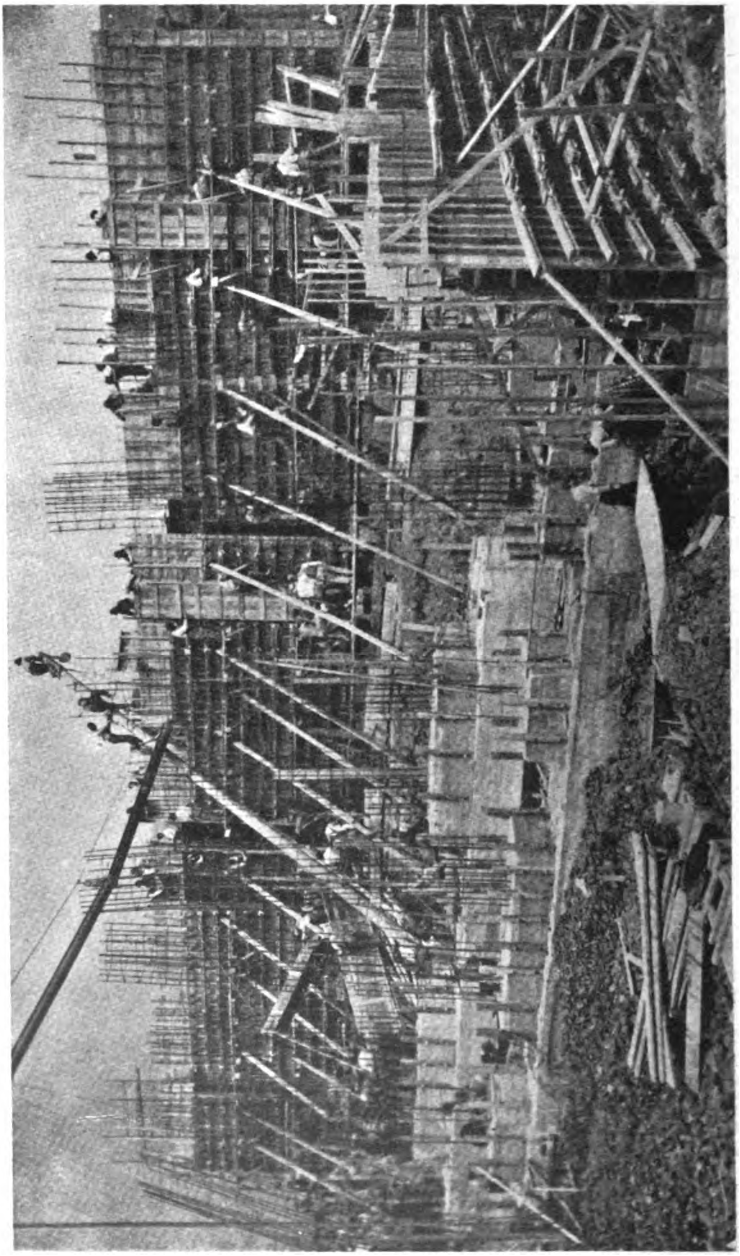


Fig. 18.—An Example of American Concrete Construction.

and the place of deposit, buggies are always employed, as they permit larger quantities to be handled at each journey with less fatigue to the workman. Transport by hand generally, however, is very limited and will not be adopted if other more expeditious and economical means can be employed in a practicable manner. No dumping of concrete for subsequent re-handling by shovels is considered necessary, and when the concrete has been mixed it is elevated and kept in the air until it can be put down in the required position in the work. The author has noticed on several schemes in this country that the concrete is deposited on a stage or some intermediate point from whence it has to be shovelled again into barrows, wagons, or into the forms, thus involving re-handling, whereas the arrangements could, by a little foresight, be made to enable the concrete to travel from the mixer to the place of deposit without any re-handling.

Two examples of American concrete work where chutes were extensively employed to feed the concrete direct into the forms are given in *Figs*. 17 and 18, and the distance from the mixer to the work was nearly 250 ft. Horse-drawn concrete-carts which are sometimes used in this country are not adopted in America, as too much time is required when the distances to be covered are considerable; locomotives are always adopted, while if the distances are small then chutes are employed for transport by gravity, and other vehicles become unnecessary.

If a mobile concreting unit is required for any special reasons, a method frequently adopted is that of mounting a large-capacity petrol-driven batch-mixer on to a flat-bottom standard-gauge railway wagon, and to this are attached wagons containing aggregate and cement to give a complete unit which can operate over a considerable distance and be independent of supplies of raw materials for some time. This method is employed for underground work or work at a low level where the mixed concrete can be discharged directly into the forms or excavations without any re-handling or transportation.

Generally speaking, the methods of transporting concrete in American work can be described as being based on the principle that no re-handling is necessary, and the method adopted will be that which is particularly applicable to the conditions under which the work has to be executed.

FORM WORK.

American practice in form work does not differ greatly from that obtaining in this country, but there are one or two points which are noticeable when a study is made of the work. In the first instance, the Americans have always had very large supplies of timber available, and in consequence they do not fully appreciate the value of the material and will not practise economy in its use unless the importance of this aspect is impressed upon them. Timber is used in enormous quantities for all purposes, and in ordinary dwellings it is the almost universal material for the whole of the structure, even the roof covering being in the form of shingles. The natural tendency in form work, therefore, is to scrap many sections that could be re-used several times and adopt a type of form which is rigid and possibly extravagant in material although its use is going to be limited. This tendency is entirely due to the conditions that have prevailed in the past when timber was adopted for shacks, roads, walks, enclosures, and any work that was necessary in the first developments of a settlement or primitive town, and the trees could be felled in the locality and roughly converted with little regard to economy. When the supplies are limited, however, and a keen supervision is placed on the making of the forms, some very excellent and economical work will result. The best form work the writer came across in America was that executed on a scheme where the general foreman was an Englishman who had previously been a carpenter in this country. The natural tendency in this case was to keep down the amount of timber to the minimum consistent with good work, and great care was exercised in arranging for easy removal and subsequent re-use.

Universal clamps and other useful devices for holding the forms together are extensively employed, and these are adopted in preference to nails or screws wherever possible.

Considerable use is also made of folding wedges, and in the case of buildings constructed on the flat-slab system the methods employed for sectionising the forms for easy removal are well consid-

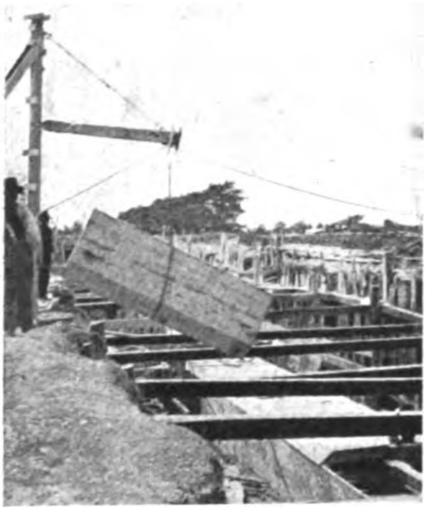


FIG. 19.—HANDLING LARGE FORMS.

ered. The design of the form work is always prepared before any of the work commences, and proper drawings are issued to the foreman in charge of the carpenters; this is the exception rather than the rule in this country.

In some schemes it is found more economical to design the forms in large sections and provide special means of handling to avoid heavy labour charges, and what is known as a "nigger-head" (*Fig.* 19) is frequently employed. This illustration shows the forms being moved from one position to another in connection with a reinforced concrete culvert constructed in a large cutting.

Metal forms are extensively adopted for circular columns with mushroom heads, which are a feature of American work, and these are usually hired by the contractor for the scheme at a specified fixed amount subject to a maximum time-clause for their return. This method is economical from the contractor's point of view, and he has the advantage of knowing beforehand exactly what the form work will cost subject to the time-clause being complied with.

Generally speaking, there is an impression of much inconsistency in American form work, as on the one hand extravagance and waste of material will be seen; while on the other hand well-designed and economical forms with efficient fastening devices are often adopted. Thus it is very difficult to summarise this section of reinforced concrete construction in a definite manner. Detailed descriptions of the forms adopted for various classes of work are not necessary here, as the information is given with explanatory drawings in several excellent American text-books, and these notes are intended to cover the practical conditions as ascertained by the writer when actually inspecting work during execution.

GENERAL.

Many of the so-called minor items receive more attention in American work than appears to be the case in this country, and new ideas are readily tried in the effort to keep up-to-date in all sections of the work, particularly in connection with labour-saving items. Wherever possible metal inserts are introduced into beams and slabs to avoid cutting away for the attachment of pipe hangers, shafting supports and similar equipment, and provision is made for vertical and horizontal pipes passing through the work by casting in proper sleeve-pieces when the concrete is being deposited. In many schemes the conduits for the electric light wiring are also cast in the work, and this represents a very large saving as compared with the cost of attaching these to the surface when the concrete is hard; in addition, the surfaces are left clear and have a better appearance. The point may be raised that it is not always possible to determine the positions of the whole of the electric light points at such an early stage in the work, and alterations will be difficult. In the case of alterations, however, it will still be possible to run a little extra conduit on the surface if this is found absolutely necessary, and it should be possible to settle the positions of the electric light points at the commencement of a scheme if the purpose for which the building will be used is known.

With regard to aggregates used, in many districts ready-mixed sand and gravel are obtainable in large quantities, and when this is the case it is the material adopted. The cement generally is of excellent standard quality, and when large quantities are purchased tests are carried out by an expert tester employed by the purchaser at the cement works, and each consignment is loaded into a box-car after his approval has been given ; the car is sealed by him so that the purchaser has a guarantee when the seals are intact on arrival that the cement is available for use immediately and no waiting for tests to be made is necessary.

The methods of dealing with tests on the concrete are similar to those adopted in this country, but some testing firms supply special boxes for taking the cement cubes, and when sufficiently hardened these cubes are despatched to the testers without removing the boxes, and if properly filled the cube will be of the size they require. The writer did not, however, find this method satisfactory, as these boxes are made of a stout cardboard only and when filled with the wet concrete they became distorted and useless for testing purposes. Metal boxes could also be obtained, but these were the subject of an extra charge, whereas the cardboard boxes were supplied free of cost if a contract for testing was placed.

A study of American methods in concreting work is interesting chiefly as regards the excellent organisation that is generally arranged and the endeavours that are made to use any plant or equipment which will facilitate the execution of the work, while new ideas are tried out in a practical manner without hesitation because there is a keen desire to keep abreast of the times and take every advantage of research and progress.

BOOK REVIEWS.

The Prevention of Vibration and Noise. By Alec B. Eason.

London : Oxford Technical Publications. Price 15s. net.

THIS is quite the best book on this little-known subject I have seen. As the author truly and modestly remarks, "even though information is published over and over again, those interested often fail to see it and waste time and money repeating tests which have already been made, or trying expedients which have already been proved to be less than the best." This remark is particularly apposite when followed, as it is, by the author's really impressive list and précis of references to previously-published work on the subject.

The author has, I think, been too modest in his title, as his book covers the measurement, nature, and effect of vibration and noise, and not only its prevention. From architects and engineers, the whole subject deserves far greater attention than it has received in the past, and this was no doubt largely due to the intrinsic difficulties of the subject, which are considerable, and to the absence of just such a volume as the author has provided us with.

The work is clearly illustrated and concisely and clearly written, and I found it a pleasure to read.

O. F.

Handbook of Cement Wares and the Artificial Stone Industry.[*] By E. Probst. 2nd Edition.

Halle a/S. : Carl Marhold. 546 pp. ; illustrated. Price 9s.

THIS useful volume, which was first published in 1919 and rapidly became known as the leading work on the subject in Germany, has now been republished in an enlarged and revised form. The enormous demand for cheap buildings in Germany has given a great impetus to the concrete and artificial stone industries there, and the objections which were formerly urged against concrete appear to have disappeared. Concrete now plays a prominent part in most building construction, and, whilst concrete blocks have a more limited application, many artificial stones, and particularly artificial marble, are now used extensively.

The subject is divided into four parts : (i) a general description of various cements, aggregates, and colours, with methods of preparation and mixing them to form concrete ; (ii) a series of descriptions of various concrete articles and the methods of producing them, particularly concrete building blocks, hollow blocks, roofing tiles, partitions, floors, pipes, slabs, stairs, piles, and masts ; (iii) artificial stones of various kinds, including granite, limestone, porphyry, and marble, and the methods of producing them; and (iv) terrazzo and mosaic work and its production. The descriptions are clear and full, and expressed in simple terms, so that anyone able to read elementary German should be able to follow them without much difficulty. Each section is arranged in a most systematic manner. There is, naturally, nothing very new or very novel, as the book is intended for practical use rather than for academic investigation, and its outlook is frankly and completely German. In many ways it is the German counterpart of *Every-day Uses of Portland Cement.*

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[*] "*Handbuch der Zementwaren und Kund steinindustrie.*"

The photographic illustrations—which are very numerous—are usually too small to be really useful, as the details in which the reader is chiefly interested are too often in deep shadow. The engravings, on the contrary, are strikingly clear, though often old. Ample tables and simple formulæ are included and their uses explained; in every way the volume is what it claims to be—a useful and reliable handbook on the subject of articles made of concrete and the allied subject of artificial stone. References to other works on the various subjects are given at the end of several of the sections, so that the reader who wishes to do so may investigate the subject still further.

Line Charts for Beginners. By W. N. Rose. London: Chapman and Hall. Price 6*s.* net.

This is an excellent little treatise on the construction and use of what are generally called alignment diagrams or nomographs, and it is doubtful if a more useful or clearer description could be given within the limits of space and of mathematical equipment on the part of the reader.

As to the usefulness of such line diagrams, the author probably has a higher opinion of them than we can confess to. Consider, for example, the example given in *Fig.* 36, p. 61, of an alignment diagram for the formula for the electrical resistance of platinoid wire:

$$R = \frac{\cdot 00005578\, l}{d^2}$$

where l is the length in cms., and d is the diameter in cms. Most people, including the reviewer, would find it quicker and more accurate to make calculations from this formula by slide-rule or logarithms, even when the alignment diagram is given. If the alignment diagram has first to be constructed, comment is needless. Apart from this, the alignment diagram is limited to lengths between 10 cms. and 150 cms., resistances between 1 and 50 ohms., and diameters between ·01 and ·1 cms. If any one of the three quantities in an example lies outside these limits, the diagram does not apply.

Then again, consider the bulk and expense of a pocket-book giving such diagrams for each metal and alloy, as compared with a table of specific resistances which is all that is required with ordinary calculations.

Then, as to accuracy, the diagram given is about 4 in. long. Certainly in practice an accuracy of 1%, in reading, corresponding to an error of $\frac{4}{100}$ inches, could hardly be exceeded. But owing to the scales having their divisions very close at one end the actual error would be much greater. Thus, near the top of the l scale the distance between 120 and 150 is only $\frac{5}{16}$ in., so that an error of reading of $\frac{4}{100}$ in. would mean an error of

$$30 \times \frac{4}{100} \times \frac{16}{5} = 3{\cdot}85 \text{ cms.}$$

approximately, or an error of about $\frac{3{\cdot}85}{120} = 3{\cdot}2$ per cent. roughly. Yet the data are given accurately to 1 in 5578, or about ·02%. This is like using a spade to prepare a microscope section, and cannot be recommended to engineers.

It is true that on p. 63 examples on this diagram are given, with results accurate to the fourth figure, but these results were obviously worked out by formula and then inserted in the diagram (see, for example, whether the figure ·02362 could be read from the diagram) which does not add to one's feeling of confidence in their usefulness.

O. F.

THE VAAL RIVER BARRAGE.

OWING to increasing demands for water in the Rand, the Parliament of the Union of South Africa, in 1914, passed an Act empowering the Rand Water Board to impound in the Vaal River a quantity of water sufficient to provide a supply of 20,000,000 gallons per day, at a total cost of about £25,300,000. In addition to several reinforced concrete reservoirs, the undertaking included the construction of the barrage (illustrated herewith) at a cost of £382,000. The Chief Engineer to the Board is Mr. W. Ingham, M.Inst.C.E., while Mr. J. C. Hawkins, M.Inst.C.E., is Resident Engineer of the Barrage.

The site of the barrage was chosen because there was a good solid rock bottom, and the fall of the river was such that the necessary quantity of water could be impounded with a depth of only 25 ft. at the barrage. The rock at the barrage site is an outcrop of amygdaloidal andesite, or diabase, which is very hard, dense, and brittle, and weighs about 142 lb. per cu. ft. The weight of the concrete is about 138 lb. per cu. ft.

The barrage consists of a series of steel gates installed between concrete piers. There are 35 piers and 2 abutments, each 8 ft. thick, 55 ft. long and 34 ft. 6 in. high. There are 36 steel gates, each 25 ft. high, the clear opening being 30 ft.; the gates are 32 ft. 6½ in. wide, being let into the piers for a distance of about 15 in. The weight of each gate is 26 tons, but with the balance weight, operating gear, and other appurtenances the total weight for each opening is about 100 tons. The pressure on each gate is roughly 300 tons when the reservoir is full. The average velocity during high floods is about 9 ft. per second.

To enable the gates, which with appurtenances weigh about 29 tons each, to be lifted, the gate is counterbalanced with a weight just over double that of the gate, and the travel of the counter-balance is only one-half that of the gate. On the top of the piers a superstructure consisting of a reinforced concrete platform supported by reinforced concrete trestle columns is built. The gates and counterbalances are suspended from this platform, and the operating gear for the gate is also worked from this level. The height of the platform from river bed level is 71 ft., and the total height from the bottom of the foundations is about 82 ft. To open a gate it is only necessary to withdraw a clutch, when the counter-balance raises the gate to within 2 ft. of the full lift. The operating gear is arranged with a flywheel, which damps the speed of the gate, and, in addition, a special cam fixed on the winding drums acts as an automatic brake at the end of the upward travel. The gate is lifted for the last 2 ft. of its height by means of worm gearing operated by hand, and is also closed by hand by the worm gearing. The time taken to open each gate is about three minutes, and to close it about twenty minutes.

The Union Government has availed itself of the opportunity of constructing a road bridge on the down-stream side of the piers, at a cost of about £22,250.

The work may be divided as follows:

(*a*) Excavation into the banks of the river to increase the waterway.

(*b*) Concrete apron 4 ft. thick, 100 ft. wide and 1,376 ft. in length.

(*c*) Concrete piers 34 ft. 6 in. high, 8 ft. thick and 55 ft. long.

(*d*) Reinforced concrete low-level bridge for crane track and footwalk. This crane is for handling the steel gates, which with accessories weigh about 29 tons.

(*e*) Thirty-seven reinforced concrete trestle columns to carry 36 spans of high-level bridge.

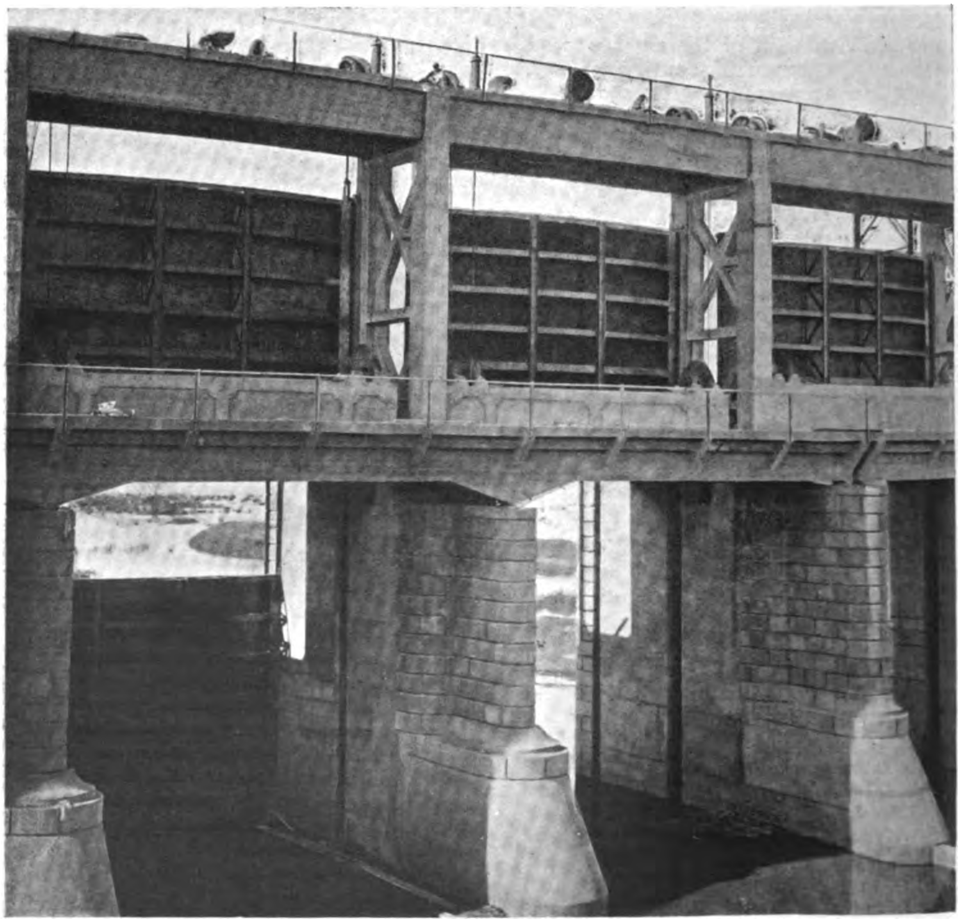
(*f*) Thirty-six reinforced concrete balance boxes 32 ft. long by 4 ft. 6 in. wide and 8 ft. high, having a total weight of about 62 tons each.

(*g*) Thirty-six steel gates each 32 ft. 6 in. wide by 25 ft. high, together with all the accessories.

(*h*) Reinforced concrete road bridge 1,400 ft. long and 36 ft. above the level of the apron.

Generally the principle adopted in the river diversion was to dam off sections of the river bed by coffer dams, commencing at the Transvaal side and working south. Concrete walls were constructed for the up-stream side and clay core-walls for the cross and down-stream walls. The latter walls were cheap and easy to replace, and in large floods were washed away, and gave a free way for the silt being carried down the river. The up-stream concrete wall was not built continuous, but gaps were left in it into which concrete slabs 6 ft. wide and 2 ft. 3 in. deep could be fitted, allowing the gaps to be opened or closed at will, so that in the event of a freshet coming down the river some of the slabs could be taken out to prevent flooding if concrete work was in progress. To enable work to proceed during freshets, cross-walls were built at frequent intervals to prevent the flooding of the whole area.

The concrete apron covers the rock bed for the entire length of the barrage and for a width down-stream of 100 ft. Its purpose is to prevent erosion of the foundations and down-stream of the piers, and for this reason is carried 45 ft. beyond the back of the piers. The average thickness of the apron is 4 ft. and the concrete mixture was 1 : 3 : 6, except for the top 6 in., which was 1 : 2½ : 5. Drains were constructed under the concrete down-stream of the gate sills to prevent any upward pressure on the apron. Whenever possible the concrete was put in for the full thickness of the apron, and one day's concreting varied from 100 to nearly 300 cu. yds. About 25 per cent. of displacers, or plums, were placed in this concrete. In order to bring the concrete to a good surface the shuttering for each day's work was accurately levelled, and when the concrete was brought up to surface level, it was rolled



Vaal River Barrage: Showing Mass Concrete Piers, Reinforced Concrete Road Bridge, Superstructure carrying Gate-Operating Gear, and Portions of Panelled Reinforced Concrete Balance Boxes.

with a 4-in. pipe, 23 ft. long ; the smallest dimension for the patch was always kept 2 ft. less than the length of the pipe, so that the pipe could be rolled along the top of the level shuttering or screens. This method of finishing off proved very effective and cheap, as it not only gave a level surface but squeezed out excess water and laitance. On the down-stream edge of the apron a dwarf wall of 3 ft. high has been constructed to form a pond at the back of the gates and thus break the velocity of water when the gates are being opened ; cross walls 3 ft. high have also been built from the end of every fourth pier to the dwarf wall erected at the down-stream edge of apron so as to prevent cross-currents.

There are thirty-five piers and two abutments, built at 38-ft. centres. The piers are 8 ft. thick, 34 ft. 6 in. high and 55 ft. long. The foundations for the piers were carried at least 12 in. into the solid rock below the apron foundation, and the concrete was of 1 : 2½ : 5 mixture. All the piers were built up in 6-ft. lifts, with 6 by 3 ft. wooden shutters, and these proved to be a very useful and handy size. For the first lift the shutters were braced to the apron, and, to provide for the successive lifts, bolts were cast into the concrete 10 ft. apart and 6 in. below the top of the shuttering. This method left a clear opening between piers for the discharge of floods, and enabled the form-work to be put up and dismantled at all times, providing of course that the river was flowing below the level of the last batch of concrete.

All reinforced concrete is at least 35 ft. above the level of the apron. The whole of the materials for the reinforced concrete have been proportioned on Mr. Duff Abrams' method for the "design of concrete mixtures," * and at present the proportions used are 1 : 2, 64 : 4, with a water ratio of 0·72 cu. ft. of water to 1 cu. ft. of cement and a relative consistency of 1 : 10. All crushed stone is saturated with water during crushing, so that the only water used subsequently is for the hydration of the cement. The sand does not absorb much moisture, as it is almost pure silica. Where the reinforcement is close together and there is little space for concrete, as on the tension side of the beams, a slightly more aqueous mixture is used in order to facilitate the filling, and the stone is screened through a ¾-in. sieve.

The low-level bridge consists of two lines of straight girders 37 ft. 10 in. long, 4 ft. high and 16 ft. thick, each weighing 14 tons, and with a clear span of 30 ft. These girders are designed to carry a "Goliath" crane capable of transporting a 30-ton load. A smaller girder weighing 7 tons will carry one side of a foot-bridge, and the other side is carried by one of the 14-ton girders. The smaller girders were all cast on the bank and after two months were placed *in situ*, and the larger girders cast on top of them. This was done to save the expense of building a gangway to carry the big girders. It was found that the large girders required a setting time of six weeks before they could be lifted without showing any deflection.

On account of the rapid filling of the small quantities of concrete required for this work, it was found that the pressures would be too great if a whole form were filled to its full height in one day unless the forms were strengthened considerably, and this would considerably increase the cost of the false-work. Owing to there being plenty of concrete work to go on with, these forms were filled in two stages at intervals of twelve hours between each, and at the end of seven days the false-work was removed. The only factor which prohibited the removal of the forms in less time was the large cross-beams required for bracing.

The false-work for the concrete for the high-level bridge is carried on trussed timber beams (35-ft. spans) at a height of 70 ft. above apron-level. These timber trusses are designed to carry 16 tons with a deflection of 1 in. and are carried on eight steel pipe supports each 27 ft. high. The total load of wet concrete and false-work per span is 64 tons. All false-work is designed with a view to rapidity of erection, dismantling and re-erection, and for use many times over without being unduly damaged. It was found that four sets of shuttering were sufficient for the thirty-seven columns, with occasional renewal of the boards.

The balance-boxes were cast *in situ* as soon as the false-work for the high-level bridge was struck. Two of the trussed beams used for the high-level bridge were then lowered to the piers and the false-work for the balance-boxes erected on them. Each box has six compartments with 5-in. outside walls and 5-in. partition walls. The compartments are each 4 ft. 2 in. by 2 ft. 8 in. by 7 ft. 3 in. deep. These compartments are filled with stone and sand to make up the requisite weight for balancing the gates. The angle iron used in the design of the false-work was subsequently used for the hand-rail stanchions on the high-level bridge.

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* See *Concrete and Constructional Engineering*, April and May, 1922.

The steel gates will be of the free roller type and will be operated by hand in single units. A complete gate consists of eight steel bowstring trusses, each 31 ft. 7 in. long by 4 ft. 6 in. deep in the centre, spaced and joined by ½-in. steel skin plates. Each gate weighs 26 tons, and has a travel of 34 ft., or 7 ft. 4 in. higher than the highest known flood. The operating mechanism consists of :—(1) A balance box weighing about 62 tons, which is attached to the gate by wire ropes ; (2) Conical drums around which the wire ropes attached to the balance box are wound ; (3) Operating headstocks. To raise the gates it is only necessary to withdraw a clutch, which allows the balance weight to come into action. The design of the mechanism is such that the maximum accelerating force is exerted at the commencement of the travel of the gate. Once movement has been started, the velocity continues to increase until the gate has travelled through a distance of from 12 to 15 ft. according to the setting. Thereafter, on account of the conical shape of the winding drums, the velocity commences to diminish and continues to diminish until the gate comes to rest. When the gate reaches its highest position it has a tendency to reverse its direction of motion and pawls drop into ratchet wheels and lock the gate in position.

The Public Works Department wagon bridge is constructed in units of four spans to the type of designs of the Public Works Department. The shuttering is carried on three trussed beams, and the total weight of concrete and falsework is about 52 tons per span. The base on which the bridge is carried is faced with concrete blocks, which were cast in moulds on shore. Blocks were also used for the up-stream nosings of piers. They weighed from 90 to 1,200 lb. each and specially hard faces were made by means of a plasterer's steel float. The material forming the body of the blocks was in the ratio of 1 : 3 : 6, and the concrete was well rammed and made with very little water. The top ¼ in. was of sand and cement, also made fairly dry, and as much pressure as possible was put on at each stroke of the float. The moulds in this case were removed in from twelve to twenty-four hours.

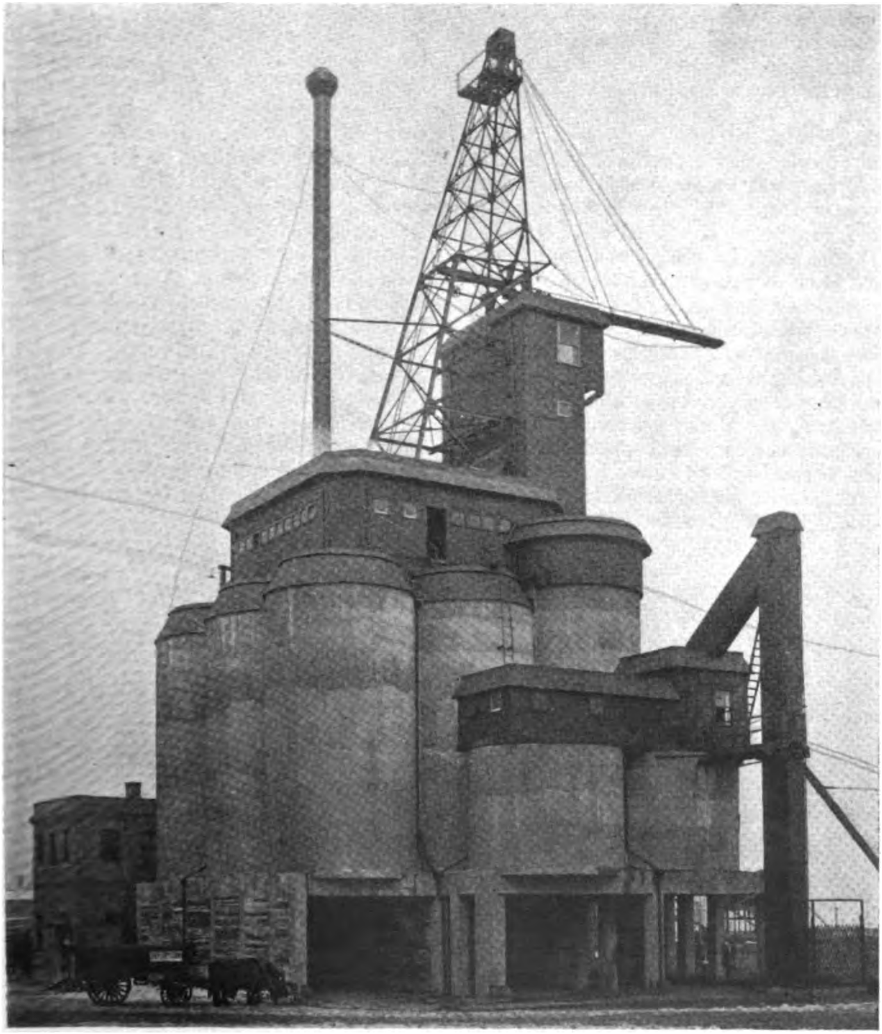
A Concrete Town in Germany.

In the course of an interesting article in a recent issue of the *Liverpool Courier*, the Cologne correspondent of that paper states :—" It is depressing in the extreme to read that in England hard-and-fast rules of so many bricks per day, etc., are combining to frustrate all efforts at a solution of the housing problem. Here in Germany, since 1919 and in some cases before that year, houses have been springing up with something akin to the rapidity of mushrooms in the night. One of the best examples of this work is to be seen at the town of Weisdorf. After the war a vast housing scheme, on a scale that would stagger our councils at home, was put in hand. The scheme involved nothing less than the building of a complete new town of a larger size than Weisdorf itself. Up to date the houses in Weisdorf—a town with a population of well over 100,000 (reliable statistics are not available)—are double the number existing there when the war ended. These houses have been constructed partly in brick, but chiefly in concrete. Houses have literally been turned out of a mould. The designs are by no means monotonous, and the architecture is varied. Every house has its own garden, with every convenience, including electric light, provided. Weisdorf two years ago was a hotbed of Bolshevism, and during the Kapp " Putsch " the workmen got out of hand, and a Soviet was actually proclaimed. To-day Weisdorf is one of the most orderly towns imaginable."

AN INTERESTING CONCRETE COAL-CYLINDER PLANT.

THE accompanying illustration shows a novel concrete coal-cylinder plant, built with moving forms in New York City. Besides the concrete coal-pockets, the plant includes coal-hoisting machinery, headhouse and appurtenances, and the scale-house. The concrete coal-pockets are formed by nine cylinders, each of 18 ft. inside diameter, on a reinforced concrete platform of columns, girders, beams and slabs. There are also two additional concrete cylinders of the same diameter and similarly supported, intended as a rescreening plant.

The supporting structure consists of four rows of reinforced concrete columns 18 ft. apart, each row consisting of from seven to nine columns. These support a



REINFORCED CONCRETE COAL-CYLINDER PLANT.

platform of beam and slab construction supporting the cylindrical pockets at such an elevation as to give 12 ft. clear headroom above the street level. Between each of the four rows of columns there is a driveway. The sea-wall affords a foundation for the row of columns nearest the river, and for each of the other three rows a foundation is provided, consisting of wooden piles driven to refusal, cut off below water level, and each row capped with a concrete wall and footing extending to within 2 ft. of the yard level and reinforced with steel near the top. After the completion of the platform, circular forms to a height of 4 ft. were erected for all nine cylinders, the plan being to cast the cylinders in a continuous operation by moving the forms up steadily as the concrete was poured.

The walls of the cylinders are of concrete 7 in. thick reinforced with 1 in. and ½ in. steel bands, spaced at varying intervals from the bottom to the top. The forms were constructed 4 ft. high, and consisted of two outer and inner rings, each ring being made up of three thicknesses of 2-in. lumber sawn to the proper circumference and bolted together to break joint. Each pair of rings (the outer and the inner) was lagged with 2-in. lumber 4 ft. long, the ribs being located 2½ ft. apart vertically and the lagging projecting correspondingly. Between the outer and inner forms of each cylinder, at regular intervals, in a vertical position, were placed six 1-in. round steel bars, with the lower ends on the platform. These rods were about 20 ft. long and each extended through a heavy yoke made up of structural steel members. Each yoke extended across the inner and outer form, with a down standing leg of angle iron 4 ft. long. A screw-jack was seated on each yoke, and by gripping each rod afforded means of lifting the forms. A concrete hoisting tower 70 ft. high was erected at one side, with a batch mixer located in a pit at the foot of the tower.

A continuous pouring of concrete in all cylinders was maintained, and the band reinforcement was placed at the proper intervals as the concrete rose. The band reinforcement was secured to the vertical rods by wires. While some workmen poured the concrete, others turned the jacks and raised the forms. Great care was taken to give each jack the same turn and to keep all forms moving together and plumb. Concrete was poured continuously for two shifts of 8 hours in each 24 hours with an interval of 4 hours between each shift, during which time the forms were kept moving by two men, who walked back and forth giving each jack a turn. The work proceeded at the rate of about 6 ft. in each 24 hours, the forms, wheeling platforms, stanchions to carry the electric lights, etc., being carried up continuously by the movement of the jacks. At night the work was lighted by electric lamps hung from timbers.

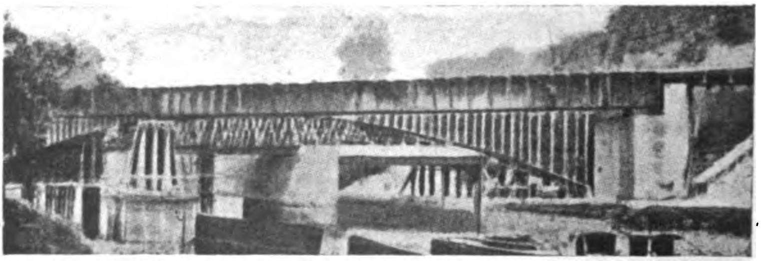


FIG.—3. CONCRETE ARCH OF 120 FT. SPAN ON THE ERQUELINS-CHARLEROI LINE.

STANDARDISATION IN BRIDGE BUILDING. (*See p.* 327.)

STANDARDISATION IN BRIDGE BUILDING.

M. LEINEKUGEL LE COCQ has recently written a most interesting paper on the reconstruction of the railways in the invaded parts of France. The railway engineers were careful in planning and systematising their schemes, and the rapidity with which they have carried on the work of reconstruction has been remarkable.

So far as the bridges built of reinforced concrete are concerned, three distinct types were recognised and used : (i) Bridges of reinforced concrete, with an arch ; (ii) Bridges of reinforced concrete on concrete supports ; (iii) Bridges of metal, with a concrete roadway. The work was greatly simplified by all the services and authorities acting in unison, and so enabling a large amount of standardisation. Thus, it was agreed that all the roads, with very few exceptions, should be confined to three sizes, viz., 10 ft., 18 ft. 6 in., and 20 ft., thereby eliminating all the pre-war troubles due to varying widths of road.

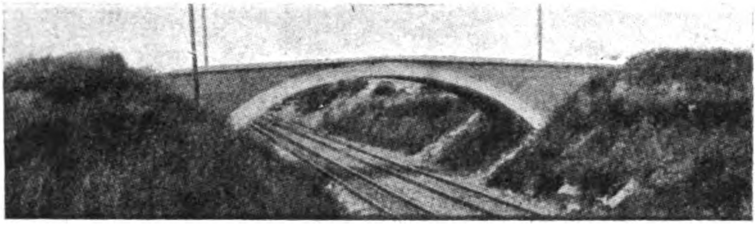


FIG. 1.—SEBASTOPOL BRIDGE, ON THE AMIENS-TERGNIER LINE.

Twenty-four bridges of the first series have been built by the Compagnie du Nord, the one shown in *Fig.* 1 being typical. These bridges were built on standard centres, made in segments so as to be suitable for various spans and capable of being used repeatedly. No less than 101 bridges of the second series have been built by the same railway company, that shown in *Fig.* 2 being typical. Only 16 bridges of the third class have, as yet, been built by this company.

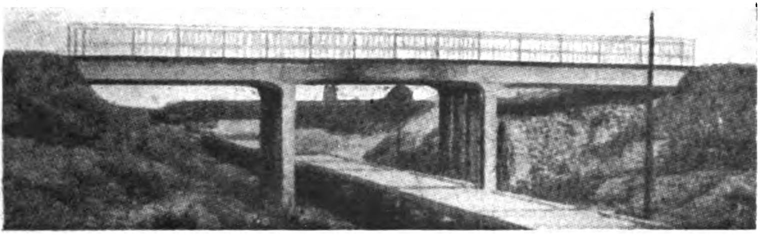
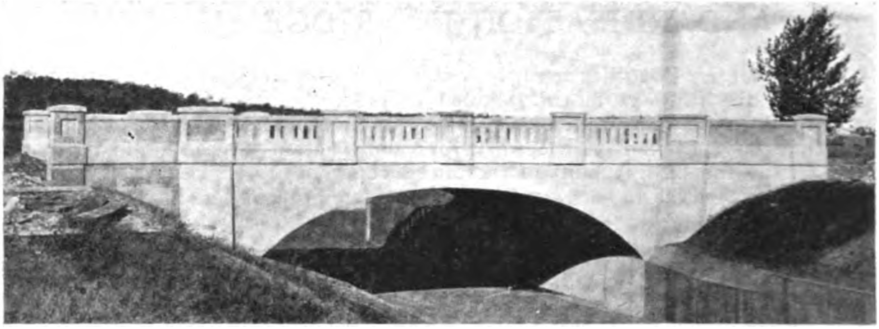
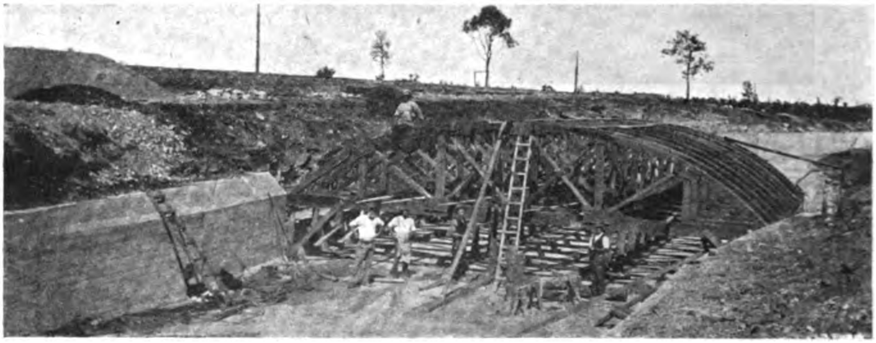


FIG. 2.—CONCRETE BRIDGE AT MERICOURT JUNCTION.

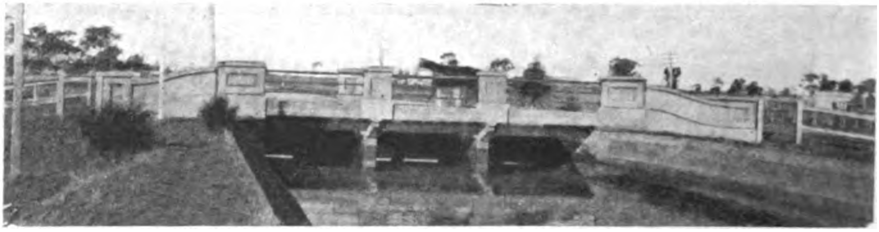
In the area covered by the Chemin de l'Est no less than 382 bridges of more than 10 ft. in length were destroyed by the Germans ; of these, 196 have now been rebuilt. The bridge shown in *Fig.* 3 is double the span hitherto used in France for a railway bridge. Its successful construction opens out a new field in the construction of railway and road bridges. The outstanding feature of these bridges is the use of standard centres, which save an enormous amount of timber otherwise necessary for this kind of work.—*Le Genie Civil.*



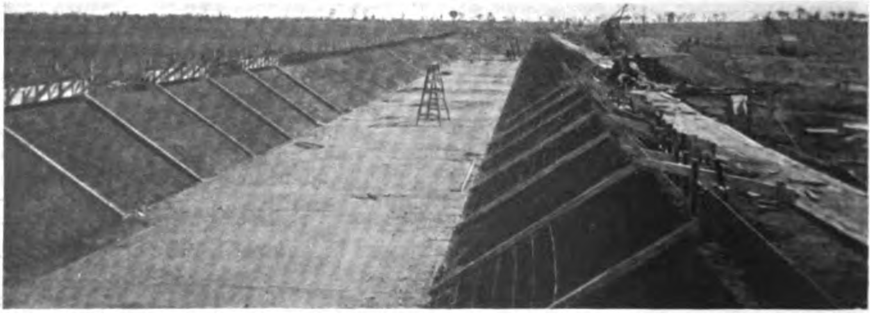
Reinforced Concrete Arched Bridge.



Arched Bridge during Construction.



Reinforced Concrete Bridge and Regulator.



Concrete Lined Canal.

CONCRETE ON THE CANAL AT GRIFFITH, N.S.W.

CONCRETE ROADS.

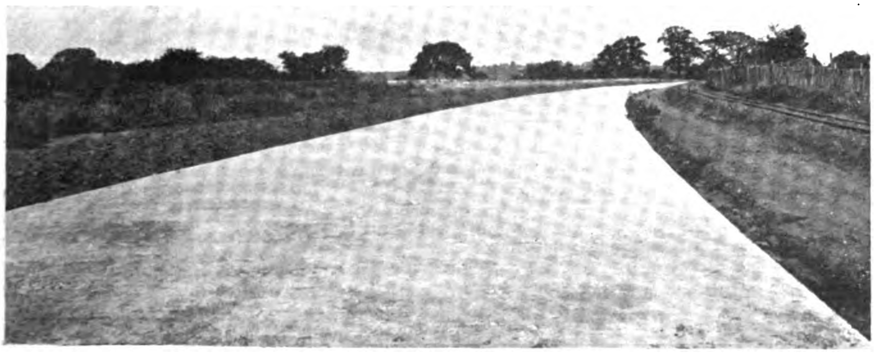
By F. WILKINSON, M.Inst.C.E.

Roads form a primary element in the material advancement of a nation, being essential to the natural resources of the country. In the laying out, construction, and maintenance of main highways the skill and ingenuity of the engineer are needed more to-day than they ever have been. The Romans were great constructors of roads and regarded them as of vital importance for the maintenance of their empire, and this is proved substantially by their road construction having in some instances borne the traffic of 2,000 years without material injury.

With the considerable increase in the use of mechanically-propelled vehicles, together with the rise in axle weights of such vehicles, the problem of providing suitable main highways which will comply with all the modern, and probable future, conditions is no light task for the engineer, and it is doubtful whether any can successfully claim to have solved the problem. From the times when the Romans first constructed their roads many systems and materials have been tried, though it must be admitted that after these excellent roads were constructed nothing really satisfactory was attempted until the eighteenth century.

At the present day concrete roads, both reinforced and unreinforced, are coming into more general use in this country, and this class of construction, both as regards material and wearing properties, has now passed with a measureable degree of success through its experimental stage, and latterly the test of continual and heavy wear. Within the past few years the writer has laid down roads, either new or widened, costing £250,000, included in which is a length of about two miles of concrete carriage-way, and also roads of other types of construction. Viewing the matter broadly, it would appear in the first place that the formation of the foundation of any main highway of the customary hardcore is often being superseded by concrete foundation, either reinforced or unreinforced; and, further, the foundation of all roads in the future may have to be of reinforced concrete of a thickness dependent upon the traffic the road will have to carry. It is to be remembered that in designing and estimating for a new main road it is quite a gamble as to whether, when the work is being carried out, suitable hardcore in sufficient quantities and at the proper price can be obtained. The supply and price of the material is constantly fluctuating within wide limits as local circumstances vary.

The question of reinforcement of concrete carriage-way foundations is a



[*Mr. Alfred Dryland, County Engineer of Middlesex;*
Mr. F. Wilkinson, Engineer to the Willesden District Council.]

Reinforced Concrete Arterial Road at Willesden.

very complex and much discussed one, and without going deeply into the matter it would seem that new foundations should be very carefully examined as to their nature, drainage, and bearing capacity. Any reinforcement should be placed in the concrete in such a position as the engineer may himself decide for any particular length. Thus, the position for steel reinforcement might for certain lengths be either at the bottom or the top of the concrete, the position being decided upon after careful consideration of the particular length under survey.

Opponents of concrete carriageways may suggest that undue length of time may elapse before the concrete is thoroughly set and traffic allowed upon the carriageway, but the writer is of opinion that before long the chemists, who by their research work help very materially in solving many problems of engineers of the present day, will have found a cement which, though costing no more and losing none of the strength of the present-day cement, yet will set so that traffic may be allowed upon the concrete with safety within two or three days, whereas six weeks is now the ideal time allowed for setting.

Although the author is well satisfied with the wearing properties of the concrete carriageway which he has already laid, it might be pointed out that it would be a wise precaution to adjust the levels of the top of a concrete carriageway so that if circumstances or traffic alter in the future a "carpet" of asphalte, tar macadam, or even wood-paving could then be laid upon the top of the concrete. The life of the concrete surfaced carriageway in this country is not definitely known, and perhaps it would not be unwise to allow for a carpet to be used at the end of the life of the concrete surface, because the cost of resurfacing the concrete would perhaps be of little less cost, comparatively, than putting on, say, 2 in. of asphalte or 3 in. of tar-macadam, and should the road have become sufficiently important wood-block paving could be utilised.

Concrete kerbs should, of course, be constructed at the same time as the carriageway, and form, if possible, an integral part of the carriageway ; where, however, up-to-date machinery is used in the construction of the concrete carriageway, this is not always possible.

The surface of a concrete carriageway appears to give a good grip not only for horse but also for mechanical traction, and the friction between the concrete and, say, a rubber tyre of a motor-car is probably much less on a concrete surface than upon a macadam or gravel road, while the grip is sufficient.

In designing a concrete carriageway it is most important that the engineer should ensure that the concrete carriageway will not be disturbed or interfered with in after years by providing as far as he possibly can for gas, water, telephones, electric cables and all other services under pathways, and provide services of such capacity to last for all time, and also to make adequate allowance for any sewers, drains, or services crossing under the concrete carriageway in conduits.

It is not proposed within the scope of a short article to deal with the much-discussed question of cracks in concrete, but the author would venture to give his opinion as to the method to be employed in order, as far as possible, to eliminate cracks, and he suggests that the work be carried out in alternate bays of not more than 30 to 35 ft. in width for a period of one week, at the expiration of which the gangs are brought back and the remaining bays then concreted in. The period of one week is suggested in order to facilitate plant work. By this method disorderly and irregular transverse cracks which appear in some continuous-laid concrete carriageways become less obvious and the joints form straight transverse lines. So long as concrete shrinks in setting—and the range of temperatures in this country is great and constantly varying—there is the danger that concrete laid as a carriageway will eventually crack, and therefore it is suggested that joints purposely made at regular intervals would be less unsightly.

Most engineers will agree that, although concrete has been used for many years, this form of construction has, until recently, not received the care to which it is really entitled. The old-fashioned type of ganger on constructional works in England holds a very high place in the estimation of engineers for his reliability and general excellence in carrying out works, and it is to be regretted that this type of man appears to be dying out. But many otherwise good foremen and gangers do not seem to be able to grasp fully the care with which reinforced concrete work should be carried out. Some of them appear to have the idea that so long as the steel is in the concrete "somewhere" then their task is carried out beyond criticism, and the author has at times pointed out to excellent men of this class working upon the construction of reinforced beams or rafts on the job that had the work been in steel it would have been made at the steel mills with the most scrupulous care and finest accuracy, together with analytical tests. Therefore the author would impress in the strongest possible manner that any work and materials used in connection with reinforced carriageways should be of the very highest standard, and no reasonable care, trouble, or expense should be too much to ensure that the whole of the work carried out should be of the very best quality possible. Anything short of this is not giving the method of construction a fair trial, and therefore if such work is carried out in a haphazard and careless way it is quite possible that the results may be anything but successful, in which case the materials or system are sometimes condemned, whereas it is the method of their employment which should rightly be blamed.

DEFECTIVE GERMAN CEMENT.

Discussing the report of the Controller and Auditor-General of the South African Railways and Harbours, a Johannesburg paper (quoted by *The Engineer*) states:—

"To meet the urgent requirements of the Administration, the High Commissioner in July, 1920, placed orders for 25,000 casks of cement with German firms, and delivery in South Africa was made in the following year. Included therein were 5,000 casks of a brand 'F,' which, on being tested in South Africa, was found to be so far below the strength shown on the specification submitted by the High Commissioner that the chief civil engineer debarred its use in any work subject to shocks or strains, and instructed that any engineer to whom it was supplied should be made aware of its character. The general manager informed me that the quality of the 'F' cement was reported to the High Commissioner, who replied that his inspector 'carried out tests as to tensile strength, and furnished a test certificate.' The general manager also informed me that similar tests made at East London showed that this cement was of a lower standard, although, on the other hand, the Government analyst reported that the cement complied with the British standard specification as regards chemical analysis. The general manager added: 'On arrival in the country, our highest technical officers condemned it for use on certain works, but did not wholly condemn it for general use, so that the purchase is not a loss in actual fact.' This cement was to be used principally for harbour construction, but the chief civil engineer has not allowed its use on marine work, and has given instructions that it may be used only 'for concrete foundations for houses, for cement plaster, and generally on maintenance work, except for bridges and reinforced concrete foundations.' The cost of this imported cement was 26*s.* 1*d.* per cask of 400 lb. South African cement—including haulage to Durban—costs approximately 14*s.* 9*d.* for a similar quantity, or 11*s.* 4*d.* less per 400 lb.

"Owing to many defective casks—of the 25,000 ordered—considerable breakage occurred on the voyage. A marine inspector was instructed to report on one consignment received at Durban, and he stated that the loss was altogether due to the casks being too weak for a long sea voyage, and unable to stand the weight of the contents. The total loss was £457 10*s.* 2*d.*

"The Director of Stores and Shipping in London reported: 'Unfortunately, against my better judgment, we were compelled, in order to assist you to bring down prices of cement, to enter the German market for supplies under Indent No. 249.G, and although the contract was very favourable as regards prices and delivery when it was made, the suppliers' attitude was most unsatisfactory through the period. Several consignments of casks were rejected before casks which were in any way acceptable were put forward. I am satisfied that it was impossible to obtain better casks from these people at the date of shipment, and as the shippers obtained a clean bill of lading for all consignments, I regret to have to advise you to abandon your claim in this instance.'"

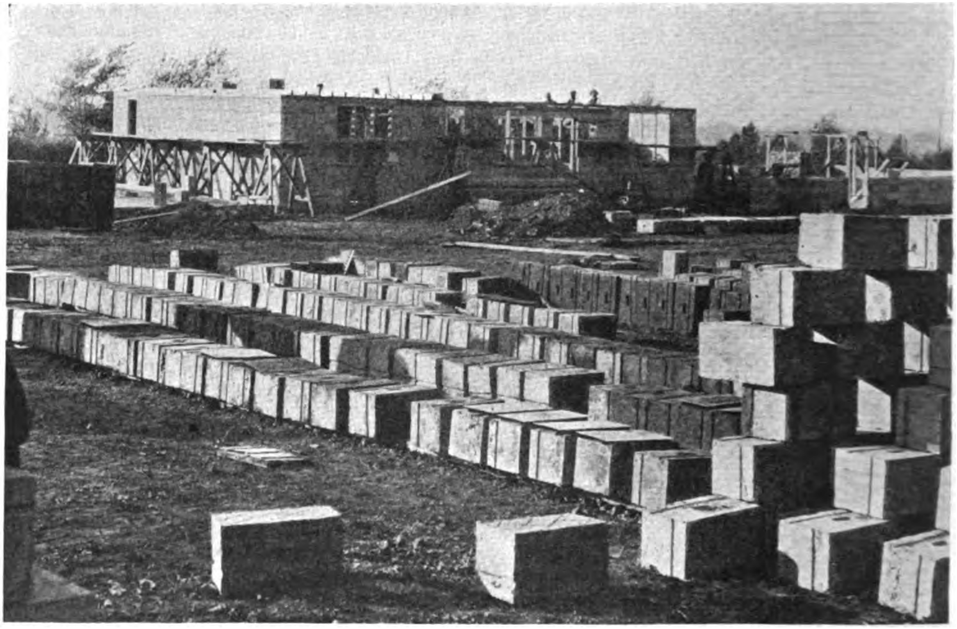


FIG. 2.

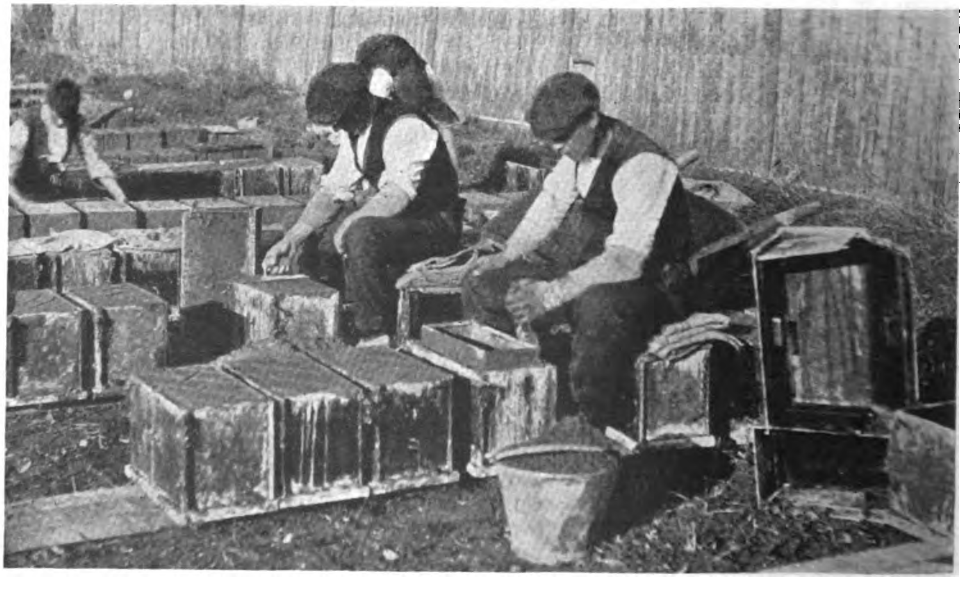


FIG. 3.
THE "DIXON-SUTCLIFFE" HOLLOW CONCRETE BLOCK (*See p.* 333).

THE "DIXON-SUTCLIFFE" HOLLOW CONCRETE BLOCK.

By OSCAR FABER, O.B.E., D.Sc.

THE number of different concrete blocks is almost as the sands of the seashore, and they all—according to their inventors—bristle with virtues and improvements. My staircase was at one time wearing thin with people showing me a really new concrete block—now I never hear of them, in spite of all their virtues. And, in truth, all these many blocks seemed to me to differ very little from one another, and all suffered from the same serious objections.

It therefore requires a little courage to write now of a new concrete block, and I only do so because the block of which I write really does seem to me to make a notable advance and overcome some of the objections of concrete blocks as commonly made and used.

It will clear the air at this stage to state briefly the concrete block problem, the usual methods adopted, and the defects in such methods.

If concrete blocks are made of concrete of such a wetness as is ordinarily used in first-class work they need to be left in their moulds at least a day. In other words, to produce, say, 400 blocks a day requires at least 400 moulds. These moulds are expensive, and as the blocks have still to be lifted and "set" in the walls by masons it is found cheaper not to use blocks at all, but to pour the wall between centering in the first instance.

To overcome this defect of expensive moulds, block-making machines were introduced. In nearly all these the principle is the same. The machine contains a mould with cast-iron sides which fall away on moving a lever. The concrete is made with much less than the usual wetness, so that if tamped hard in this mould the sides may be removed at once, and the block, though, of course, quite weak, will stand up. The block has been built on a wooden or metal bottom, which is then carried away to the storage and curing yard, while a new block is made on a new bottom.

In this way the cost of moulds is reduced to that of bottoms (instead of whole moulds) to which must, of course, be added something for the use of the machine. It should also be noted that the number of bottoms required must be considerably in excess of the daily production of blocks, because with this process the blocks are not strong enough

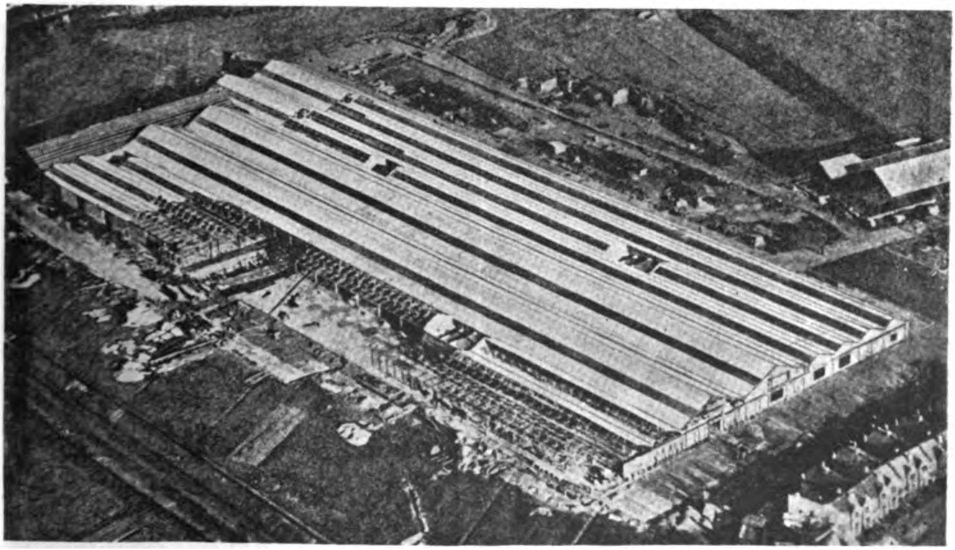


FIG. 1.

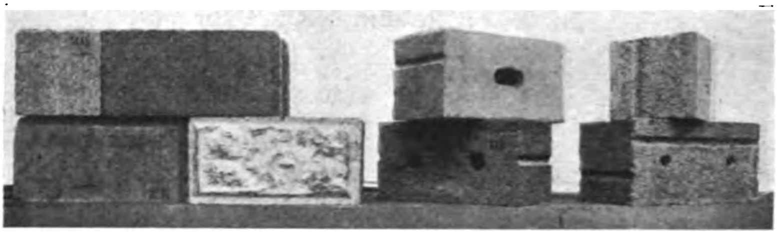


FIG. 4.

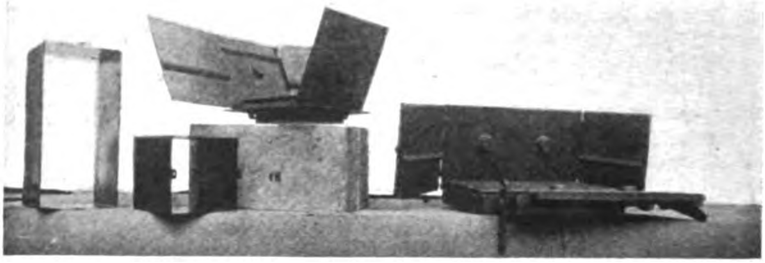


FIG. 5.

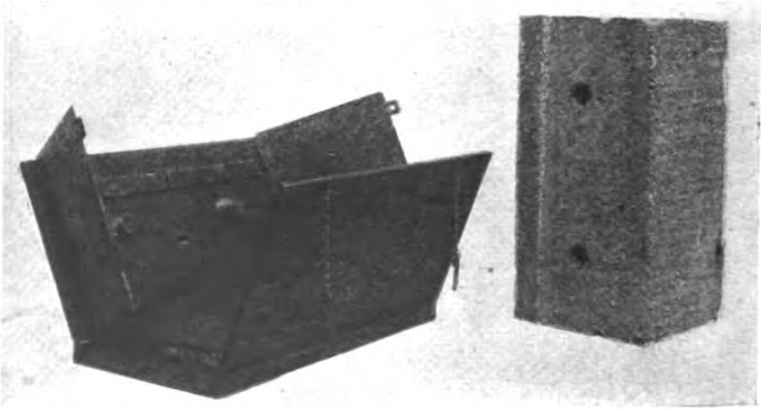


FIG. 6.

THE "DIXON-SUTCLIFFE" HOLLOW CONCRETE BLOCK.

to lift off their bottoms the day after moulding, except in exceptional circumstances.

We will not here go into the question whether blocks so made are cheaper than concrete poured direct into walls between centering after due allowance has been made for the cost of setting the blocks by masons. The important matter which concerns us first is whether blocks so made are good. A considerable experience with them brings out the fact that they suffer from the following defects :—

(1) They produce a concrete of only a proportion of the strength of concrete mixed with the *best* percentage of water, which, however, is too wet to enable the mould sides to be removed at once without damage to the block.

(2) The concrete produced is less watertight than concrete made with the *best* percentage of water.

The air-spaces left in hollow blocks are necessarily small—owing to the need for thick walls if the blocks are to be moved when newly moulded. This makes for heavier blocks and more expensive handling, as well as more material, which is also expensive.

In regard to these defects, the writer has often found blocks made under apparently good conditions of supervision, etc., having less than one-fifth of the strength of the same concrete mixed wet in the ordinary way, and, as for porosity, the use of these blocks has often led to complaints which can only lead to concrete being given a bad name by people who do not realise it is not concrete as such which is at fault, but concrete improperly made. In one instance, the writer has seen a jug of water poured gently on to a side of such a block and the whole jugful absorbed into it in the space of five minutes. When it is remembered that the same proportions of concrete properly mixed with water would resist,

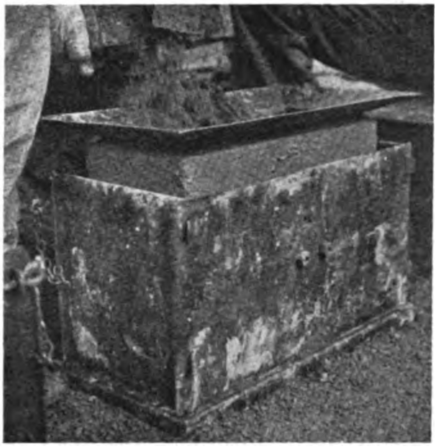


FIG. 8.

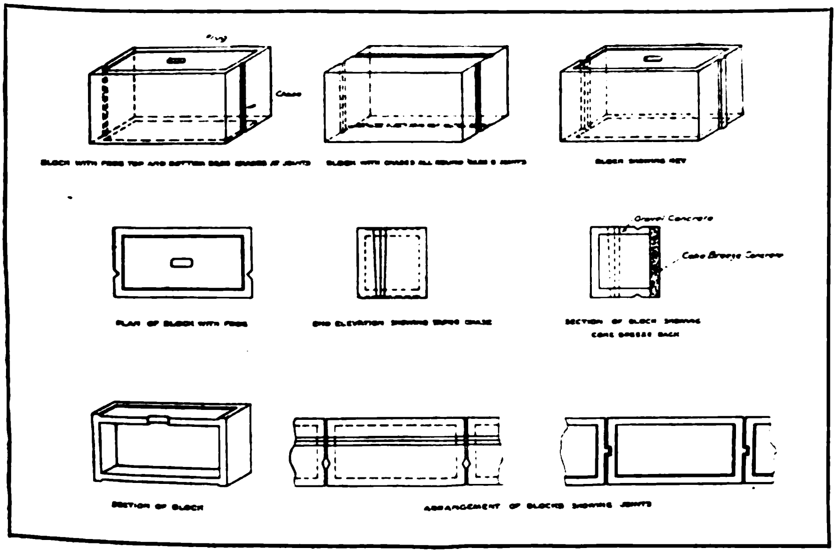


FIG. 7.

say, 10 ft. head of water on a 6-in. thickness without the slightest percolation, it will be seen what a different material is produced.

Practically all the "new and improved" blocks previously referred to were of this type, and differed only in a slight change of form, or some detail in the machine, all the essential defects remaining.

It must not be understood from the foregoing remarks that concrete blocks made in the semi-dry manner have no sphere of usefulness—on the contrary, in spite of being porous and not so strong as wet-mixed concrete of the same proportions, they are generally strong enough for what is required, and porosity does not in some applications constitute a defect, and where it does it can be remedied by rendering, or by double-wall construction. It is a fact, of course, that for cottages, etc., such blocks have been used with considerable success. Nevertheless, it must be granted that, where strength and watertightness are important, they are much inferior to wet-mixed concrete.

With the "Dixon-Sutcliffe" block no saving of moulds is effected, but saving is made by getting first-class concrete, and so being enabled to make a big reduction in the thickness of walls, saving cement and ballast, and saving labour both in block-making and in block-setting owing to the lightness of the block. These blocks are generally 18 in. by 9 in. by 9 in., and the walls are about 1 in. thick. Clearly with such thin walls, difficulties would be found at the bedding of one block on the other, both as regards strength and making the joints weatherproof. This difficulty is overcome by providing the blocks with a top and bottom, also about 1 in. thick.

Practical people will ask how such a block is constructed—it savours rather of how to get the apple into the dumpling! The construction is really very ingenious. An inch of concrete is first laid in the bottom of the mould. On this a thin sheet-steel box, open top and bottom, is stood, so that it stands 1 in. from the mould sides all round, and its top is 1 in. below the mould top. The space inside this is filled up with sand, and the space around with concrete, up to the level of the top of the inside box, which is now pulled away. The top inch of concrete is then finished, resting partly on the concrete walls and partly on the sand core.

When the block has been set a day the outer mould may be removed for re-use. The sand can be removed by leaving a hole in the top and bottom when casting.

A feature of these blocks is the fine sharp corners and edges which good concrete makes possible, and in regard to strength and imperviousness they clearly have all the properties of concrete at its best.

The accompanying photographs show applications in practice. *Fig.* 1 shows the Central Overhaul Factory for the London General Omnibus Company at Chiswick. The magnitude of this work is clear from the photograph. *Figs.* 2 and 3 show the application to cottage construction. *Figs.* 4 to 7 show details of blocks and moulds. *Fig.* 8 shows a block having the sand core filled in through a funnel. It is interesting to note that the blocks have been tried with success in the East as well as in this country.

Further particulars regarding this block may be obtained from the proprietors, Messrs. Hollow Concrete, Ltd., of 70 Victoria Street, S.W.1, or from Professor Dixon, of the City and Guilds Engineering College, South Kensington. My reason for drawing attention to this block is my anxiety that anything possessing such real merit should not be overlooked.

The Road Congress at Seville.

IN our next issue we hope to give a full report of the papers on concrete roads to be read at the Fourth International Road Congress now being held at Seville. From the preliminary agenda which has reached us it appears that the all-concrete road will receive a great deal of attention at this congress.

"Concrete Roads."

WE have to announce that the first edition of "Concrete Roads" is now out of print. A second edition, considerably enlarged and brought up-to-date, will be ready in June.

MEASURING SANDS IN AN INUNDATED CONDITION.

In view of the difficulties encountered in producing a fairly uniform concrete in the field owing to the bulking and water-retaining properties of the fine aggregate, a new method for measuring sand in an inundated condition, which seems to indicate the possibility of much closer and more constant measurements than can be obtained by present methods, has been devised by Mr. R. L. Bertin, Chief Engineer of the White Construction Co., of New York, and is described in the *Proceedings* of the American Society for Testing Materials.

The principle on which this method is based is that if sand containing any percentage of moisture is inundated by dropping it into a vessel containing water, so that the vessel is filled to the top with sand and water, the amount of sand and water therein contained will be nearly constant so long as the sand has a constant void factor.

A device was designed for the purpose of measuring simultaneously predetermined amounts of sand and water accurately and consistently. This device consists of a container shown in *Fig.* 1, subdivided vertically in two sections, the upper section (*A*) is used to measure inundated sand, and the lower section (*D*) to measure water. Both sections are made adjustable as to capacity by bolting to the bottom of *A* a combination of rings (*E*) of different heights, so that for a concrete mix requiring a given quantity of sand and water the two sections can be so adjusted as to contain the proper amount of sand in *A* and the right amount of water required for a concrete batch, the total amount of water being made up of that amount of water which the sand in *A* will hold, which is practically equal to the voids in the sand, plus the amount of water held in *D*. The bottom (*C*) of section *A* is provided with screened openings as shown in section *AA*, which permit water to enter into and exit from section *D*, but exclude sand from entering it.

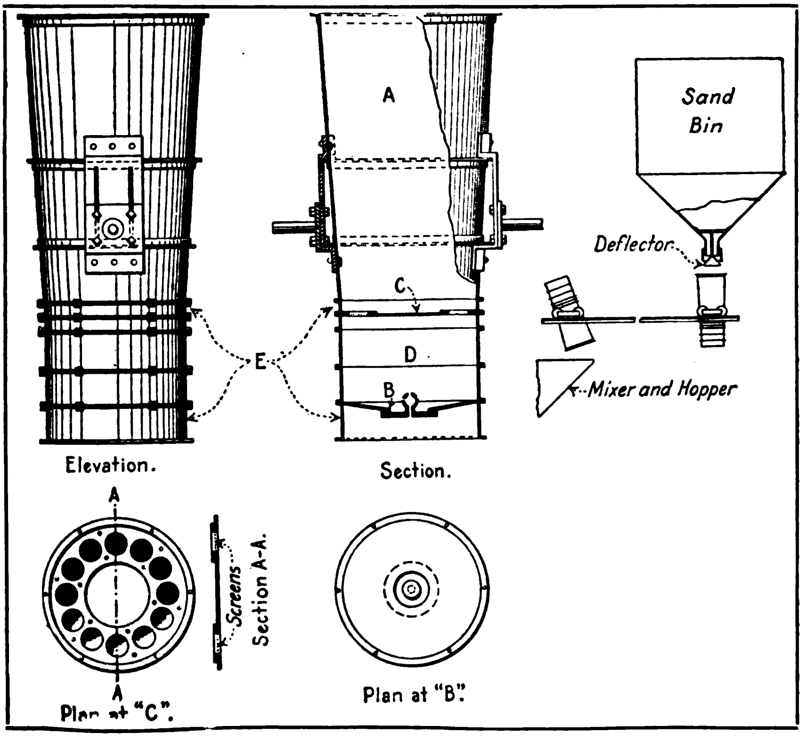


Fig. 1.

The operation of the device consists, after the capacities of the sections have been adjusted, in filling section *D* and approximately ⅛ of section *A* with water, then filling section *A* with sand. The container is then emptied into the mixer hopper delivering to the mixer definite quantities of sand and water for each batch.

In order to determine the constancy of the concrete made with the use of this device, several tests were made. Five one-bag batches of concrete were mixed in a power mixer in the proportions of 1 bag of cement, 1·77 cu. ft. of sand (measured in an inundated condition) and 3·60 cu. ft. of gravel. Two batches were made with dry sand, one batch was made with damp sand containing approximately 6 per cent. of water which for the sand used is the percentage causing maximum bulking, and two batches were made with wet sand containing approximately 14 per cent. of water. In every batch the sand in the condition mentioned was poured in water previously put into the measuring device and the sand so measured together with the water it held was emptied into the mixer hopper, and for all batches 22 lb. of water were added.

From the determinations noted, the volume of concrete per batch, the water-cement ratio, the cement factor, the weight of cement per batch and the weight of inundated sand per batch were computed; slump tests were made (from 6 by 12-in. cylinders), 6 by 12-in. cylinders were cast and tested at twenty-eight days. The similar determinations for the five batches were averaged and curves were plotted showing the percentage every one of the batches varied above or below the average.

Curve No. 1, plotted for the volume of concrete per batch, shows that the amount of concrete produced was practically the same for every batch, notwithstanding the difference in moisture content of the sand in the different batches.

Curve No. 2, plotted for water-cement ratio, indicates very slight variations when consideration is given to the fact that a uniform amount of water was added to every batch in spite of the varying amount of moisture contained in the sand for the different runs. The variations in this curve are due partly to the variations in the weight of the cement contained in the bags used in the different batches. The weight-of-cement curve varies inversely as the water-cement ratio curve; had the weight of cement been constant, the water-cement ratio curve would show less deviation from the average.

The cement factor curve indicates very slight variations from the average.

Curve No. 5 shows the variations in strength, and Curve No. 7 shows the variations in slumps. While the strength and slump vary to a greater extent than the other factors, the results may be considered fairly constant for such tests.

Curve No. 6 shows the variation in amount of sand and water contained in 1.77 cu. ft. The variations are small and are not sufficiently large materially to affect the volume of concrete per batch as indicated by Curve No. 1.

The results indicate the possibility of producing, by the use of the device described above, fairly constant concrete, no matter what the condition of the sand is relative to moisture content.

Strength of Reinforced Concrete Bridges.

No comment is necessary on the following excerpt from a London daily newspaper—it simply shows that when put to the test reinforced concrete will fulfil the claims made for it: "Nine trucks were smashed to pieces yesterday against the buttress and archway of the station bridge newly erected to serve the British Empire Exhibition at Wembley. The accident was caused by the derailment of one truck in the centre as the train climbed a stiff gradient. It dragged the other trucks with it, and all ran away down the incline, tearing up sleepers as they went. The trucks travelled half a mile before they crashed into the bridge. So jammed were the wrecked trucks that they occupied less than 45 ft. of rail space. The bridge, which is of ferro-concrete, stood the severe strain so well that it shows no more damage than could have been wrought with a sledge hammer."

MAKING CONCRETE FORMS.

(*Contributed.*)

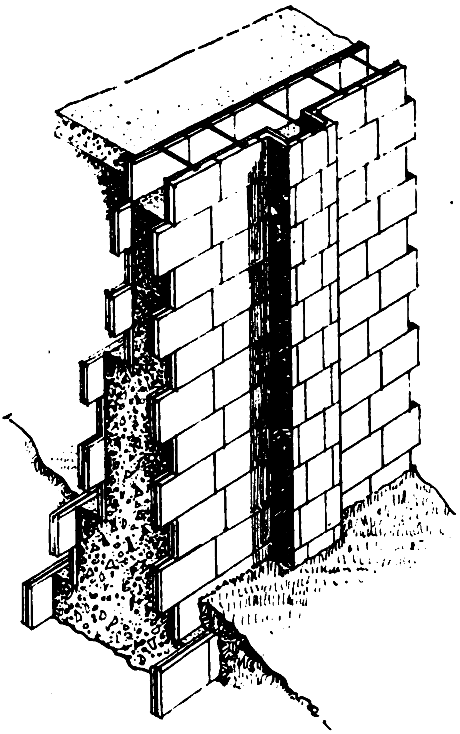
Probably the greatest disadvantage in the universal use of concrete as a structural material is that connected with the making of the forms required before any kind of monolithic wall may be constructed. Generally the operation of building forms occupies much the greater part of the whole period of constructing a wall, the cost usually being wholly out of proportion to the ultimate value of the wall itself. That is to say, all the labour and material used in the making of forms for concrete walls is ultimately rejected, no part of it adding to the intrinsic value of the wall. This is especially the case in walls such as are used in house construction, where the outlay on the forms—even when used a number of times over—is frequently as costly as the wall itself.

Repetition in the use of forms is to-day almost an unknown factor. The number of times that they may be employed depends largely on the design of the wall itself, and most of all on the workmanship and materials in the forms. Wooden forms can seldom be "standardised" except in exceptional cases of mass production; but such work is the exception rather than the rule, and in any case methods of construction should not depend upon wholesale production, but rather upon the adaptability of any particular method to any particular condition in which it finds itself.

Amongst the chief drawbacks to forms, either wood or metal, is the fact that they do not always produce a wall of a perfectly finished and workmanlike surface, certainly not one acceptable from the architectural standpoint, which is so important a consideration to-day. To make it presentable, therefore, plaster has invariably to be used to cover up the multitudinous blemishes which appear on the surface of form work, however finished it may be.

There is no reason why concrete itself in certain thinly cast forms should not be used in the making of concrete walls, and so replace all temporary wood and metal forms. The sketch illustrates the general principles of these concrete forms. A series of thin, finely-mixed and rodded cement-sand slabs or tiles and angles, locked and jointed by a simple arrangement of tongues, grooves, and ties, may be used as the forms themselves. There must be two units of construction—the ordinary flat tile for the flat surfaces, and an angle-tile for all angles and returns.

These tiles may be made with a deep interior corrugated surface, which forms a grip for the poured concrete, and thus they get homogeneously keyed into the mass, from which they become inseparable. The vertically-rodded mixture which is most suitable for thin pieces produces a slab which is very much harder than any that can be obtained by hand or hydraulic pressure, or even by vibration, jigging, or rotation. The tiles may be made as thin as 2 in., and of a weight not exceeding about 40 lb. This would give a length of about 1 ft. 9 in. and a height of 12 in. Thus, these thin slabs may be handled with as much facility as wood, and be set up very quickly with fine joints to present ultimately a perfectly finished stone-like



Concrete Slab Formwork.

effect. Not only do the tiles form an integral part of the structure, but being so finely made the wall becomes a finished product on both faces at one operation, and the attendant labour of building and removing forms is dispensed with altogether.

The accomplishing of this simple operation, together with a number of outstanding economies, effects considerable savings all round. For instance, a wall required with an effective thickness of 12 in. may be made up of only 8 in. of poured concrete, the slab forms making up the remaining 4 in., thus effecting a saving of poured concrete of 33⅓ per cent.

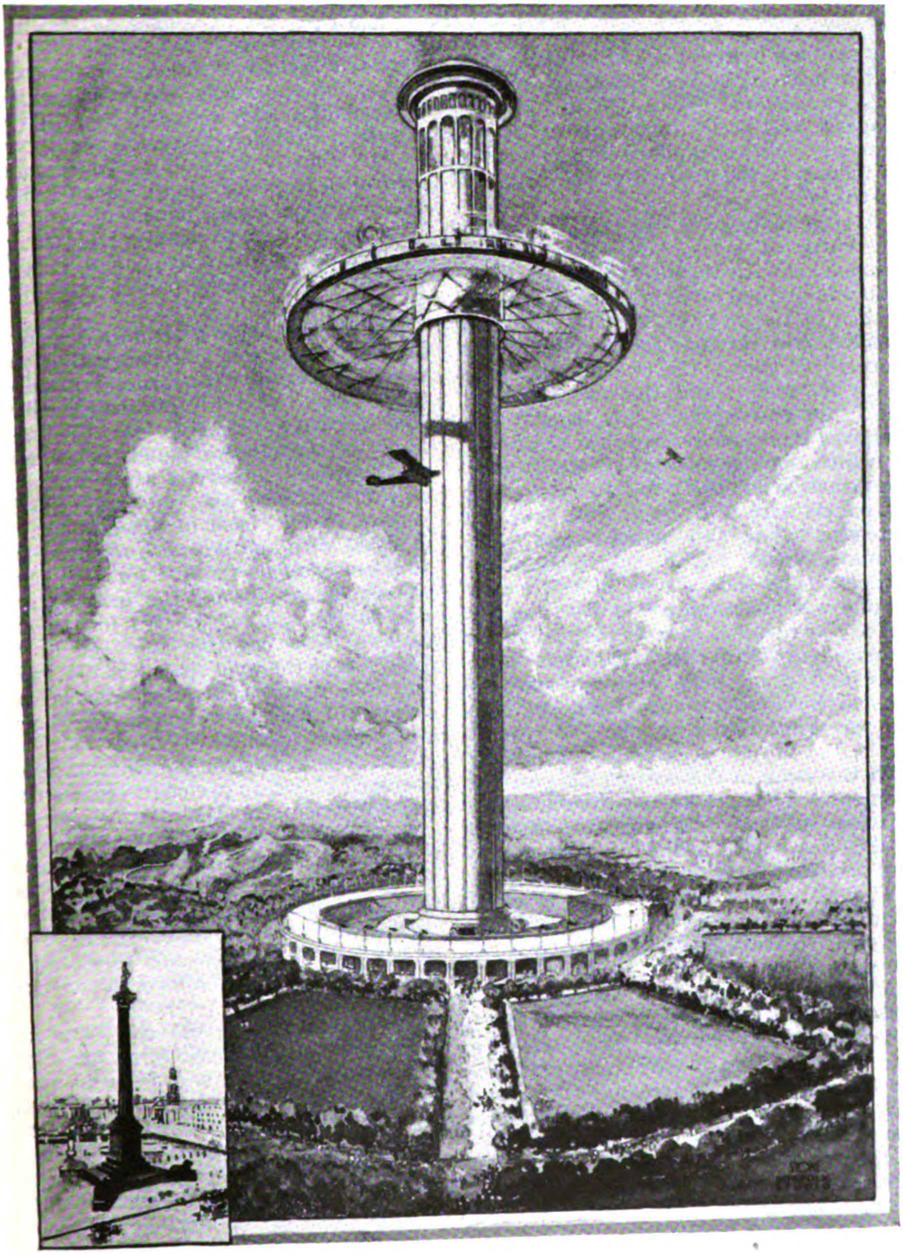
Since the slabs can be easily cut to form closers at any given point, there is no necessity for "specials." The width of the wall is regulated by the length of the metal tie, which stays the slabs in position both ways. These ties may also very conveniently be utilised for wiring on the vertical or horizontal reinforcements, as they occur at every one-foot of height and about every 18 in. of length. The ties may be twisted in the centre so that the poured concrete in its descent into the wall cavity will not unnecessarily jar the general structure.

The cost of these slab forms works out at about 9d. per ft. super erected, and taken with the saving effected in the poured concrete, and the elimination of plaster the total cost of the forms for any wall would be not more than that required in wood or metal forms.

A high mixture should be used in the composition of the slabs, not less than one part of cement to four parts of washed and sifted sand, which will produce excellent surface finish.

PROPOSED AMUSEMENT TOWER AT WEMBLEY.

On the opposite page we illustrate the design for a reinforced concrete tower which it is proposed to erect in the amusements section of the British Empire Exhibition, Wembley. The tower would be 500 ft. high and 45 ft. in diameter. The wheel, shown three-quarters of the way up the tower in the photograph, would be 200 ft. in diameter and carry 400 passengers ; the outer rim of the wheel would be mainly glass, giving passengers an opportunity of seeing the surrounding country and the exhibition premises from the air. The wheel would rotate six times in travelling from the top to the bottom of the tower, the journey taking three minutes. When at the top the wheel would rotate twice in one minute and then make six turns in the descent, the whole journey occupying seven minutes. The wheel would have a covered-way to the body of the tower, and it would be possible to land passengers at any height during the ascent and descent. There would be two lifts in the tower for the benefit of those who prefer that means of ascending, and there will be forty landing-stages inside. In all, the tower would require some 10,000 tons of concrete and would cost approximately £100,000 to build. The base of the tower would be occupied by sixty shops, restaurants, tea-rooms, etc. From the lantern, powerful coloured horizontal and vertical rays would be projected at night, and the wheel would be balanced by counter weights mounting on its own weight and half its passenger-carrying capacity, so that when fully loaded it would only be necessary to lift a dead weight of 15 tons. The engineer for the Exhibition is Mr. E. O. Williams ; and Messrs. Wembley Amusements, Ltd., of 50 Pall Mall, S.W.1, are the promoters of the scheme.



Proposed Reinforced Concrete Tower at Wembley
(*See page* 340.)

REINFORCED HOLLOW TILE AND CONCRETE FLOOR TEST.

A TEST on a hollow tile and concrete floor slab reinforced in two directions, carried out by Messrs. W. A. Slater, Arthur Hagener and G. P. Anthes at the United States Bureau of Standards, is summarised as follows in the *Journal* of the Franklin Institute :—

A large slab, 48 ft. by 115·5 ft. centre to centre of outer supporting girders, was loaded. In the 48-ft. dimension the slab was divided into three spans of 16 ft. each. In the 115·5-ft. dimension it was divided into six spans ; two end spans were 16 ft. each ; the two middle spans 19·25 ft. each ; and the two spans at the other end were 22·5 ft. each. The slab was therefore divided into 18 panels, of which six panels were 16 ft. square, six panels were 16 ft. by 19·25 ft., and six panels were 16 ft. by 22·5 ft. Girders extended along all panel edges, and columns stood at all intersections of girders. The slab was 6 in. thick over all and was made up of 6 in. by 12 in. by 12 in. clay tiles laid in rows parallel to the panel edges, with ribs of concrete 4 in. wide and 6 in. deep between rows of tiles in both directions. Each concrete rib was reinforced with one ½-in. round bar in the top for negative moment, and with the same in the bottom for positive moment. The bars for negative moment extended out into the panel one-fourth of the span length in the direction of the bar, and were there hooked for anchorage.

The slab reinforcement had a yield point of about 54,000 lb. per sq. in., and the girder reinforcement about 46,000 lb. per sq. in.

According to current methods of design the working loads (sum of live and dead loads) for the three sizes of panel would be 83 lb., 67 lb. and 48 lb. per sq. ft. for the square, intermediate-size, and long interior panels respectively. The weight of the slab was approximately 50 lb. per sq. ft.

For about two months the square panels stood under a uniform load (sum of live and dead loads) of about 450 lb. per sq. ft., the intermediate-size panels under 330 lb. per sq. ft., and the long panels under 280 lb. per sq. ft. The load was then shifted and (with smaller loads on adjoining panels) the loads for the succeeding six months on certain panels were : Square panels, 780 lb. per sq. ft. ; intermediate-size panels, 655 lb. per sq. ft. ; and long panels, 555 lb. per sq. ft. Finally, the load was concentrated still more, and for about eleven months the loads on single panels of each size have been : Square panels, 1,463 lb. per sq. ft. ; intermediate-size panels, 1,234 lb. per sq. ft. ; and long panels, 970 lb. per sq. ft.

During the loading strain gauge readings of elongation and shortening were taken at about 1,600 critical places in the concrete and in the reinforcement. The negative moments for the 16-ft. spans of the interior panels based upon these stresses (observed after maximum uniform loads of 450, 330 and 280 lb. per sq. ft. had been in place about two months) with an allowance for dead load stresses, were 0·0186 Wl for the square panels, 0·0277 Wl for the intermediate-size panels, and 0·0398 Wl for the long panels. The negative moment in the direction of the long span was 0·0213 Wl for the intermediate-size panels and 0·0179 Wl for the long panels, where l is the short span.

The co-efficients for the exterior panels averaged about 20 per cent. greater than the co-efficients for the interior panels.

Both the deformations and the deflections increased when the load remained constant over a considerable time, but the fact that so great a load has been carried for so long a time without excessive deflection removes any misgiving as to the adequacy of the form of construction used.

The deflections at the centres of the interior panels observed after the maximum uniform load had been in place about two months averaged 1/273, 1/563, and 1/1800 of the clear short span for the square, intermediate, and long panels respectively. Upon removal of 80 per cent. of the load the recovery was 50 per cent. or more of the original deflection.

At this stage of the test the largest crack was less than 0.01 in. wide. The largest crack in the under-side of the slab under a load of about ten times the design load was about 1/40 in. wide. The points of zero stress in the negative reinforcement occurred at about one-fifth points of the spans.

Under the greatest loads applied the shearing stresses in the ribs of the slab were 405 lb. per sq. in. or more, assuming the tile walls to have been effective in resisting shear. If the resistance of the tile walls is neglected the shearing stress must have been 500 lb. per sq. in. or more. The estimated shearing stress in one of the girders is 440 lb. per sq. in. The loads on which these shearing stresses are based had been on the slab for nearly a year.

OBSERVATIONS ON CEMENT.

Dr. Otto Schott has recently published a series of observations on cement based on experiments which have in some cases lasted as long as six years.

He found, for instance, that if a thin pat of neat cement is suspended in a shallow air-tight vessel of water, crystals of calcium hydrate gradually form, and in six years are clearly visible to the naked eye. This experiment clearly proves that water can decompose Portland cement, forming simple compounds and calcium hydrate, and from it Dr. Schott concludes that the hardening of cement is chiefly due to the adhesion of the particles of silicate, the spaces between them being filled with the crystalline hydrate. He considers that this explains why cements rich in lime and those which are exceptionally finely ground harden more rapidly than others. He regards expansion and the resulting cracking as due to the formation of an excess of the calcium hydrate crystals from a cement unduly high in lime.

In addition to the crystals just mentioned, Dr. Schott observed a gelatinous growth of mono-calcium silicate. A series of 168 cements was made from pure silica, pure alumina, pure iron oxide, and pure lime, the mixtures being burned in an electric furnace at a temperature high enough to effect sintering. The temperature varied with the composition of the mixture; in some instances it reached 2,000° C. with cements rich in iron oxide or alumina. Iron oxide behaved curiously, as it increased the sintering temperature of mixtures rich in that oxide, but in smaller proportions it acted as a flux.

A micro-petrographic examination of the cements showed the following: The aluminous cements contained small crystals of calcium aluminate ($3CaOAl_2O_3$). A cement composed wholly of this substance is only feebly doubly refractive, but one composed of tri-calcium ferrite ($3CaOFe_2O_3$) is strongly doubly refractive, though much less transparent. Some of the clinkers contained definite crystals with double refraction and a composition corresponding to Jänecke's compound ($8CaO, 2SiO_2, Al_2O_3$), whilst the corresponding iron compounds produced beautiful crystals with strong double refraction.

The cements rich in alumina were very difficult to work, and even when mixed with 2 per cent. of gypsum required about an hour to set. The corresponding iron cements set in ten to twenty minutes.

Compression tests showed that the strength is roughly proportional to the lime-content until the tri-calcium compounds are formed. An excess of lime then causes cracking, and consequent weakness. The iron oxide cements set very slowly and are weaker than the alumina cements, yet reached 5,600 lb. per sq. in. in twenty-eight days. The hardness was greatest with aluminous cements, but an excess of alumina is harmful, as it causes cracks.—*Tonindustrie Zeitung.*

A NEW CONCRETE PIPE-MAKING MACHINE.

Two of the factors which help to make concrete buildings cheaper than other permanent structures in most districts are that concrete made on the site is cheaper in initial cost than brick or stone for a given quantity of walling, and that the cost of transporting local aggregates is considerably less than the railway freightage on manufactured materials. This applies not only to buildings but to all other concrete products where suitable aggregate is available locally, especially now railway charges add so considerably to the cost of all bulky and heavy goods. In order to take advantage of the elimination of railway charges which the manufacture of concrete goods on the site makes possible, many machines have been produced for making such articles as concrete kerbs, flags, pipes, blocks, etc., and the increasing use of these goods is the measure of their popularity amongst those to whom cheapness consistent with a first-class product is a consideration.

We recently had an opportunity of seeing a machine capable of making concrete pipes of any size between 6 in. and 24 in. of the same pattern as stoneware pipes, including a collar. The machine is portable, a type which should appeal to local authorities carrying out sewerage or drainage works on which it is desirable that as much local unemployed labour as possible should be absorbed.

As is shown by the drawing, the machine comprises an outer mould and an inner withdrawable mould or core between which the concrete is led and tamped, thus ensuring that the internal surfaces of the pipes are smooth and cylindrical. The inner mould supports the concrete while it is being tamped, and is then withdrawn helically from the pipe so that a trowel carried at its lower end smooths the inner surface of the bore of the pipe as it is exposed.

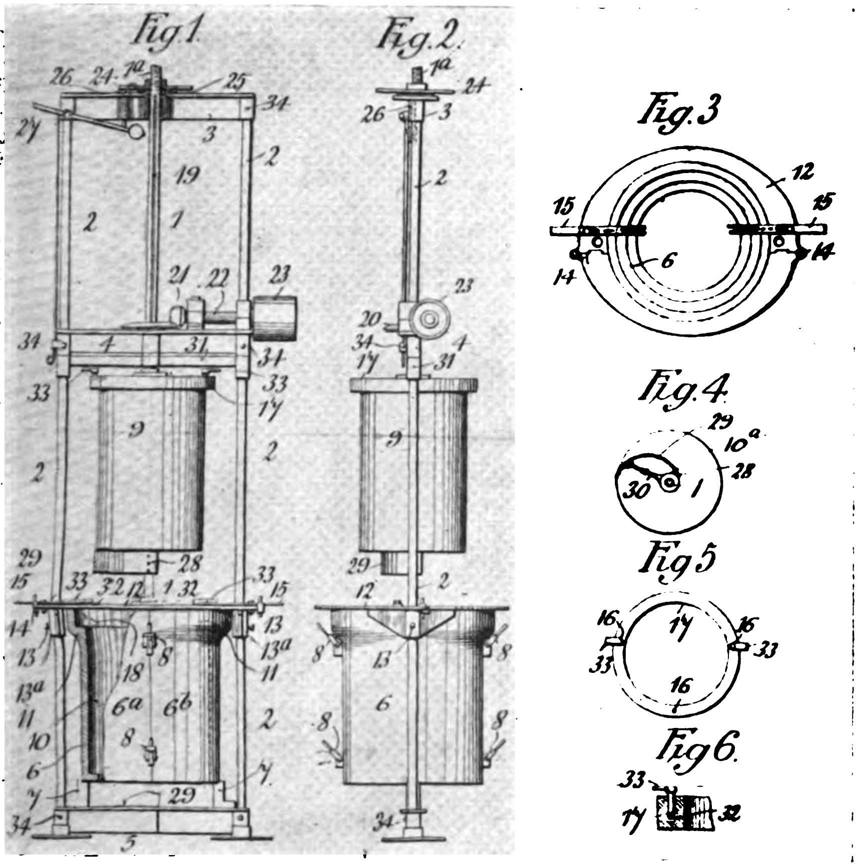
In the drawings *Fig.* 1 is a sectional front elevation; *Fig.* 2 is a side view; *Fig.* 3 is a plan of the mould with the divided annular plate or table; *Fig.* 4 is an underside plan of the inner mould or core; *Fig.* 5 is a top plan of a ring serving as a socket core; and *Fig.* 6 is a detail view of the method of causing the collar to grip the core.

The vertical main shaft (1) is journalled in a main frame consisting of two or more vertical members or standards (2) braced together by cross-beams (3, 4, 5). The outer mould (6) is supported by chairs (7) adjustably carried by the bottom cross-beam (5), and vertically divided into two halves (6*a*, 6*b*) held together by clamping handles (8). The inner mould (9) is carried by the lower end of the main shaft (1) and adapted to be raised and lowered in the space within the vertical standards (2) and between the middle and lower cross beams (4 and 5), so as to descend into the outer mould (6) or to be withdrawn therefrom. Concrete to form the moulded pipe is fed into the space between the outer and inner mould and tamped, the inner mould continuously supporting the inner surface. The top of the outer mould is enlarged (11) and embraced by an annular plate or table (12) slidable on the vertical standards (2) and capable of being secured in position thereon by set screws (13); this table (12), which is made in halves connected by hinge pins (14), carries radially sliding bars (15) adapted to engage studs (16) projecting upwards from a collar (17) thus held centrally within the enlarged part of the outer mould so as to act as a core for moulding a socket (18) on the pipe being moulded. The upper portion (1*a*) of the main shaft is externally threaded or screwed, whilst its lower portion is formed with a keyway (19); the rotation of this shaft is effected by bevel gearing. On the upper end of the main shaft, above the top cross beam, is fitted an internally-threaded head wheel adapted either to run freely on ball bearings supported by the beam, or to be held against rotation by means of a clutch or brake (26) carried and controlled by a hand lever (27); when the head wheel is thus held, the shaft and core are slowly raised out of the outer mould. Secured to the lower portion of the main shaft is a blade or trowel (29) projecting radially slightly beyond the base of the core, and which consequently, as the core is rotated and slowly raised, travels helically over and smooths the inner face of the bore; the blade is curved backward and has a certain amount of spring, and its sweep or radius may be adjusted by means of a right and left handed screw (30). A radius rod or beam (31) slides on the standards and laterally supports the main shaft near the top of the core.

The rotating motion of the inner mould brings a film of cement to the surface, and thus renders the pipes impervious to water, when a mixture of one part of cement to three parts of sand is used. The outer moulds can be withdrawn from the pipes about twenty-four hours after they have been made, and used again, so that it is not necessary to have more than about thirty outer moulds with each machine. It is claimed that a reasonable output for the machine is 200 pipes 2 ft. long, 6 in. diameter, per week, or 100 24 in. diameter pipes. As regards cost, we are informed that concrete pipes can be produced on this machine from 7*d.* per foot for 6 in. pipes up to 4*s.* 4*d.* per ft. for 24-in. pipes, which is less than half the cost of stoneware pipes of the same sizes.

In tests carried out on some of the first pipes made in August last year, a 6-in. diameter pipe 2 ft. long withstood an internal pressure of 30 lb. per sq. in., and an external load of 1,660 lb. per ft. run.

The machine weighs one ton complete, and requires from 2 to 5 H.P., according to the size of the pipe being made, and is easily operated by a man and a boy. The manufacturers are Messrs. The Mono Pipe Works, of Lower Green Road, Esher, Surrey, who are also supplying pipes made on the machine.



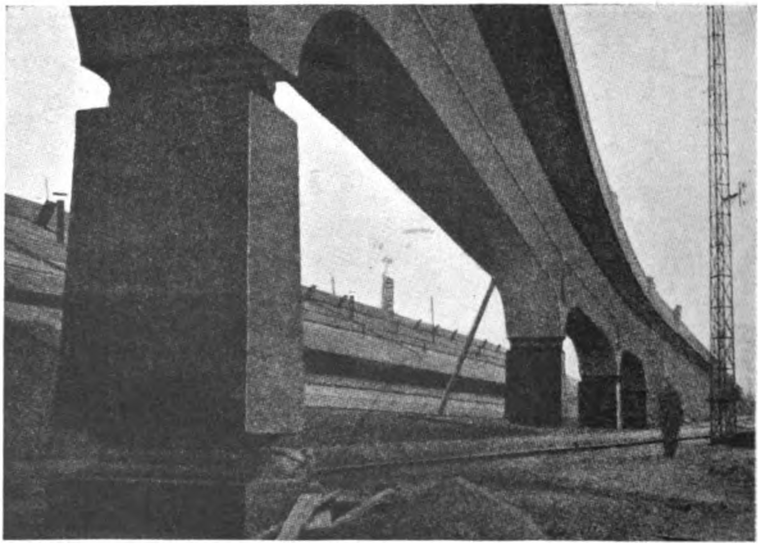
A New Concrete Pipe-Making Machine.

DESIGN OF FRAMED STRUCTURES.

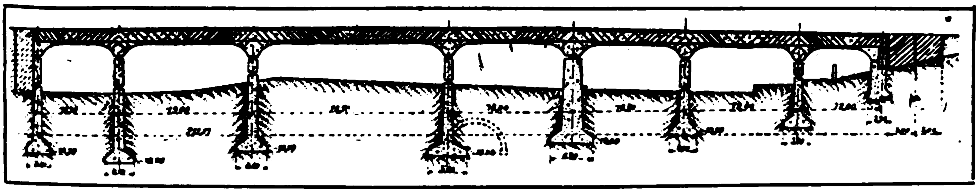
At a meeting of the Institute of Structural Engineers held on March 22, Professor A. J. Sutton Pippard, M.B.E., D.Sc., A.M.Inst.C.E., read a paper entitled " A Method for the Direct Design of Framed Structures having Redundant Bracing."

In the course of his opening remarks, he said from the standpoint of purely theoretical design all the members in a framed structure should be made of such dimensions that their material would be stressed to the allowable limit under the external load system considered, since a structure proportioned in that way would ensure maximum economy in the use of material. In many cases such an ideal structure might be impracticable, and would have to be modified to meet a variety of conditions imposed by considerations other than those of theoretical design. Whatever modifications were necessary, however, to meet such conditions, the ideal design would have considerable value as a basis for the development of the final scheme embodying the requirements of practical engineering. In some cases, e.g. in the design of all aircraft structures, the reduction of weight was such an important consideration that very little was permitted to stand in the way of its attainment, and in such cases the ideal structure would be very closely followed.

In the case of a framework which was constructed of just a sufficient number of members arranged in such a way as to render the structure simply stiff under any external load system, the internal forces in all the bars of the structure could be found by the methods of pure statics. The internal load distribution was then dependent only upon the geometry of the framework, and was not affected by the sizes of individual members or the material of which they were made. Under those circumstances ideal design was a simple matter, since the cross section of each member could be proportioned so as to develop the requisite stress. Such a structure permitted of direct design. In the case of a redundant structure, i.e., one which had more than the minimum number of bars requisite for stiffness, the internal force distribution could not be so readily analysed, since the elastic properties of the members themselves exercised a considerable influence in determining such distribution. In such a structure the forces in the bars could only be estimated by an application of the principle of least work, or by a direct comparison of strains in the members. The former method was that



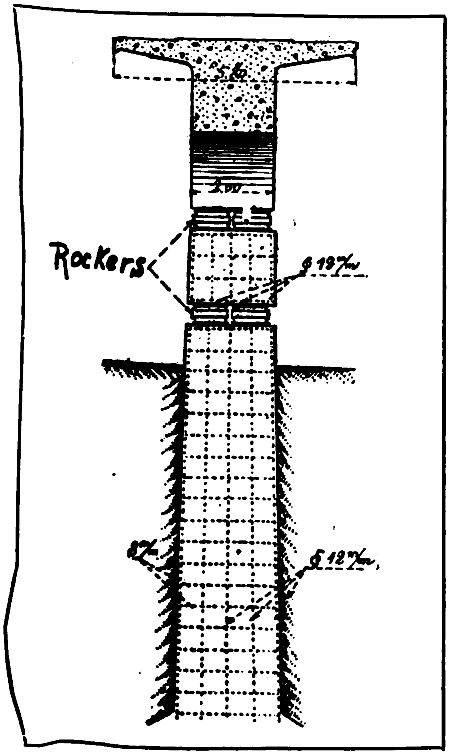
St. Chamond Bridge: Showing Rockers. (*See p.* 347.)



St. Chamond Bridge: Longitudinal Section.

most commonly in use, but both had the disadvantage that before the internal force distribution could be determined it was necessary to know the cross sectional area of every member of the structure. If such a structure therefore had to be designed cross sections must be assumed before calculations could be made. An analysis could then be carried out by either of the methods mentioned. The stresses in different members would in all probability be found to vary considerably, some being very much below the allowable limit, and others possibly above. The design might therefore either be uneconomical in material or unsafe, and a new estimate of sizes must be made and the laborious calculations again carried out. Design thus became a matter of trial and error, and the procedure for obtaining stresses entailed a very considerable amount of computation work.

The lecturer then described a new application of the principle of least work by means of which the trial and error method in the case of a redundant structure was eliminated, and the designer was able to control the magnitude of the stresses during the course of the design.



St. Chamond Bridge:
Section-Elevation of Pier.

BRIDGE WITH PENDULUM PIERS.

A new railway bridge near St. Chamond consists of seven very shallow spans having a total length of 410 ft. and a massive "table," or bridge, of reinforced concrete resting on a central pier and five other, piers. The central pier is provided with a steel rocker, and each of the other piers has two steel rockers one above the other with about 6 ft. of concrete between them. The rockers were used because of the excessive strains due to the frequent passing of trains comprising a locomotive of 50 tons and a truck with castings weighing 200 tons. Each rocker consists of a steel cylinder, ⅜ in. diameter, and two base plates 18 in. by 20 in. and 24 in. by 37 in., to distribute the load on the pillar. The axes of all the cylinders are parallel in order to prevent distortion when the concrete expands; consequently, the axes of the rockers do not necessarily correspond with the axes of the pillars. The bridge proper was designed as a simple beam 412 ft. in length.

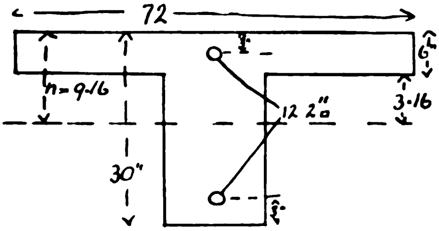
When the bridge was tested with the pre-arranged load of 200 tons and a locomotive weighing 50 tons, it was found that the deflections were well within the limits, and the distribution of the load on each of the pillars was remarkably close to what was anticipated. The elasticity of the "table" was all that could be desired.—*Le Genie Civil.*

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

***Readers are cordially invited to send in questions relating to concrete, which will, as far as possible, be answered direct and subsequently published where they are of general interest. We cannot give complete designs; these are the province of the consulting engineer. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.*—Ed.**

REINFORCED CONCRETE BEAMS.

Question.—I should be much obliged to you if you would look through the enclosed example, which is the same as the **T**-beam question in your January issue. I have done it according to the example on page 97 of Andrews' *Elementary Principles of Reinforced Concrete Construction*. You will see that I get a different neutral axis and bending moment. Should it not agree with what you obtained in the example ?—T.D.S.



Example.

Taking moments we have :

$$72 \times 6\left(n - \frac{6}{2}\right) + m\ 12{\cdot}2(n - 3)$$
$$= m\ 12{\cdot}2(30 - n).$$

Here m is 15 (assumed).

$$\text{Then } 432n - 1296 + 183n - 549$$
$$= 5490 - 183n$$
$$798n = 7335$$
$$n = 9{\cdot}16.$$

Now $B = \frac{cI}{n}$ where I = Moment of Inertia of slab about Neutral Axis.

$$I = \frac{72 \times 9{\cdot}16^3}{3} - \frac{72 \times 3{\cdot}16^3}{3}$$
$$+ 15 \times 12{\cdot}2(20{\cdot}84)^2 + 15 \times 12{\cdot}2\ (6{\cdot}16)^2$$
$$= (18500 - 756) + 80{,}000 + 6{,}950$$
$$\therefore I = 17{,}744 + 86{,}950$$
$$= 104{,}694 \text{ in.}^4$$
$$\therefore B = \frac{cI}{n} = \frac{104{,}694 \times 600}{9{\cdot}16}$$
$$= 6{,}850{,}000 \text{ in lb.}$$
$$\text{or } B = \frac{tI}{m(d - n)} = \frac{16{,}000 \times 104694}{15 \times 20{\cdot}84}$$
$$= 5{,}350{,}000 \text{ in. lb.}$$

i.e. $B.M. = \underline{5{,}350{,}000.}$

Answer.—The calculation based on Mr. Andrews' book is, of course, correct as regards method. The solution given in our January issue was only meant to be an approximate one, and the discrepancy between $n = 9{\cdot}7$ and $n = 9{\cdot}16$ is not serious. But our correspondent in his calculation has taken d as 30 in. instead of $d = 27$ in. This will reduce his moment of resistance from 5,350,000 down to very near the figure of 4,600,000 we gave.

INFLUENCE LINE—CONCRETE SURFACES.

Questions.—(*a*) What is an " influence line " ? I have looked up about a dozen books on the theory of structures and on bridge design and do not see the term used.

(*b*) It is the practice, I believe, to obtain a surface finish to concrete work by a process of bringing the " fat " of the cement to the outside by beating with hoop iron bats. It has occurred to me that this method, while having the effect of giving a specially impervious coating externally, might have the serious effect of impoverishing the concrete around the reinforcement, or at least around the hooping, to such an extent that slipping may occur. Is this a matter for serious consideration ?

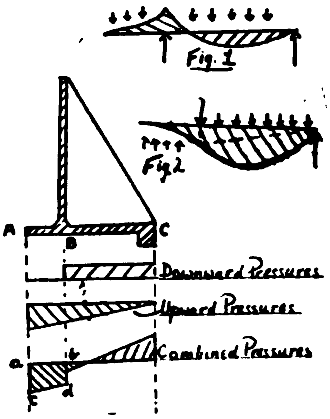
W. J. L.

Answers.—(*a*) An influence line is a curve in which the bending moment, or shear, at a definite point (such as mid-span) due to a rolling load, is set down to scale under the successive positions occupied by the rolling load. For example, the moment at midspan when the load is at A is set down under the point A, and so on.

(*b*) We do not think there is any risk of what is suggested. The extra work on the concrete has the effect of reducing voids and causing better consolidation, and seems to produce a denser concrete without the defects suggested.

RETAINING WALL.

Question.—Will you tell me what loads and moments are to be taken in the design of the base-slab of a counterforted retaining wall with heel beam? (*see sketch*). According to the method in Ring's *Reinforced Concrete* the loads and moments are as follows:—



Heel Beam

Load.—Equal to the vertical component of pull in rib and taken as equally distributed.

Moment.—$\frac{Wl}{12}$.

Slab BC

Load.—Same load as acting on rib which divided by area of slab gives load per sq. ft. and taken as equally distributed.

Moment as for beam free at C and fixed at B.

Slab AB

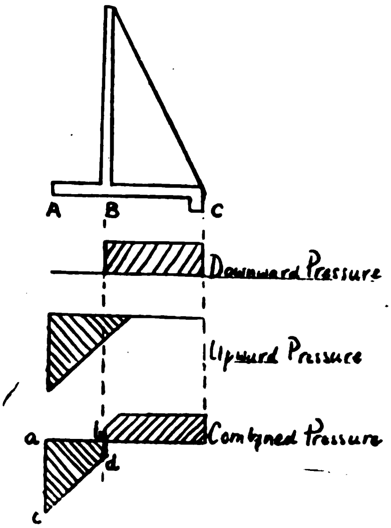
Load.—Upward pressures ***a, b, c, d***.

Moment.—That for moment of pressures ***a, b, c, d***—*moment of fixation if slab* BC *at* B.

I cannot follow this because: (1) The heel beam, I believe, takes the reactions of the back slab BC, and hence the total load on the slab must be more than on the beam; (2) I cannot see how the moment of fixation of slab BC can reduce moment due to loading on cantilever AB.

If the loading on both slabs acted in the same direction the moment would take the form *Fig.* 1, the negative moment over support being caused by the cantilever. If the loading on the cantilever were reversed, the moments, it seems to me, would be in the form of *Fig.* 2.—E. A.

Answer.—The criticism of Ring's treatment of the base-slab of a counterforted retaining wall is quite correct, and we agree with the comments, as well as the moment curves indicated. We would further point out that the pressure curve of upward pressure does not as a rule die away from a maximum at one end to zero at the other. As a rule, the resultant pressure strikes the ground outside the middle third, when the pressure curves are more like the following:



This clearly simplifies the design of the back-slab (BC), which should be treated as a rectangular slab supported at B and C in one direction and by the vertical buttresses in the other. The toe (AB) should, of course, be designed as a cantilever supporting the pressure ***a, b, c, d*** without any deduction. This cantilever moment is balanced partly by a moment in the slab BC, but also partly by a moment in the vertical slab.

PROPORTIONING CONCRETE MIXTURES.

(*Concluded from p.* 260.)

DESIGN OF CONCRETE MIXES.

THE problem of designing concrete mixes using given materials resolves itself into that of finding the combination which with a given water-ratio will give a concrete of suitable workability using a minimum of cement. The following outline will make clear the steps to be followed in the design of concrete mixes on the basis of our studies of concrete :

STEPS IN THE DESIGN OF CONCRETE MIXTURES.

(1) Knowing the compressive strength required of the concrete, determine by reference to *Fig.* 1 * the maximum water-ratio which may be used, subsequent steps in the design of concrete mixes are only devices for securing a workable concrete using this water-ratio and a minimum quantity of cement. It is obvious that a given water-ratio can be secured with a minimum of cement if the aggregate is graded as coarse as permissible (considering its size and the mix used) and if we use the driest mix which can be properly placed. Securing a coarse, well-graded aggregate, using rich mixes, employing the driest practicable consistency, using mechanical methods of placing concrete, etc., are all methods of producing a workable mix with a minimum water-ratio. Experience or trial is the only guide in determining the relative consistency of concrete necessary in the work. Obviously the driest workable consistency should be used. The size of aggregate available, or which must be used, and the other factors will furnish a guide as to the mix. The mix is expressed as one volume of cement to a given number of volumes of aggregate ; that is, the combined fine and coarse aggregate. In general, some allowance must be made for the high strengths in laboratory tests. In other words, a water-ratio somewhat lower than that given for the required strength in *Fig.* 1 should be used. For convenience in the subsequent steps we shall deal with concrete strength instead of water-ratio (as in *Fig.* 6), although it should be understood that it is the water-ratio which fixes the strength so long as we have a plastic mix.

(2) Make sieve analysis of fine and coarse aggregates, using Tyler standard sieves of the following sizes : 100, 48, 28, 14, 8, 4, ⅜, ¾ and 1½ in. Express sieve analysis in terms of percentages of material by weight (or separate volumes) coarser than each of the standard sieves.

(3) Compute fineness modulus of each aggregate by adding the percentages found in (2), and dividing by 100.

(4) Determine the " maximum size " of aggregate by applying the following rules : If more than 20 per cent. of aggregate is coarser than any sieve the maximum size shall be taken as the next larger sieve in the standard set ; if between 11 and 20 per cent. is coarser than any sieve, maximum size shall be the next larger " half-sieve " ; if less than 10 per cent. is coarser than certain sieves, the smallest of these sieve sizes shall be considered the maximum size.

(5) From Table 3 determine the maximum value of fineness modulus which may be used for the mix, kind and size of aggregate, and the work under consideration. (The values in Table 3 are plotted in *Fig.* 5.)

(6) Compute the percentages of fine and coarse aggregates required to produce the fineness modulus desired for the final aggregate mixture by applying the formula :

$$p = 100\frac{A - B}{A - C} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where p = percentage of fine aggregate in total mixture.

A = fineness modulus of coarse aggregate.

B = fineness modulus of final aggregate mixture.

C = fineness modulus of fine aggregate.

Fig. 7 may be used for solving Equation 3. and for making comparisons of the effect of certain changes in proportions of fine and coarse aggregates. The distinction between fine and coarse aggregate is solely for convenience in securing a uniform grading; the division may be made at any desired point.

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* Figs. 1 to 4 and Tables 1 and 2 were given in our last issue.

(7) With the estimated mix, fineness modulus and consistency enter *Fig.* 6 and determine the strength of concrete produced by the combination. If the strength shown by the diagram is not that required, the necessary readjustment may be made by changing the mix, consistency or size and grading of the aggregates.

(The quantity of water required can be determined from Formula 4, or approximately from Table 5.)

Important Note.—It must be understood that the values in *Fig.* 6 were determined from compression tests of 6 in. by 12-in. cylinders stored for 28 days in a damp place. The values obtained on the work will depend on such factors as the consistency of the concrete, quality of the cement, methods of mixing, handling, placing the concrete, etc., and on age and curing conditions. Strength

TABLE 3.—MAXIMUM PERMISSIBLE VALUES OF FINENESS MODULUS OF AGGREGATES.

For *mixes* other than those given in the table, use the values for the next leaner mix. For *maximum sizes* of aggregate other than those given in the table, use the values for the next smaller size.

Fine aggregate includes all material finer than No. 4 sieve; *coarse aggregate* includes all material coarser than the No. 4 sieve. *Mortar* is a mixture of cement, water and fine aggregate.

This table is based on the requirements for *sand-and-pebble* or *gravel* aggregate composed of approximately spherical particles, in ordinary uses of concrete in reinforced concrete structures. For other materials and in other classes of work the maximum permissible values of fineness modulus for an aggregate of a given size are subject to the following corrections:

(1) If *crushed stone* or *slag* is used as coarse aggregate, *reduce* values in table by 0·25. For crushed material consisting of unusually flat or elongated particles, *reduce* values by 0·40.

(2) For *pebbles* consisting of *flat particles, reduce* values by 0·25

(3) If *stone screenings* are used as fine aggregate, *reduce* values by 0·25.

(4) For the top course in *concrete roads, reduce* the values by 0·25. If finishing is done by *mechanical means*, this reduction need not be made.

(5) In work of *massive proportions*, such that the smallest dimension is larger than ten times the maximum size of the coarse aggregate, *additions may be made* to the values in the table as follows: for ¾-in. aggregate 0·10; for 1½-in. 0·20; for 3-in. 0·30; for 6-in. 0·40.

Sand with fineness modulus lower than 1·50 is undesirable as a fine aggregate in ordinary concrete mixes. Natural sands of such fineness are seldom found.

Sand or screenings used for fine aggregate in concrete must not have a higher fineness modulus than that permitted for mortars of the same mix. Mortar mixes are covered by the table and by (3) above.

Crushed stone mixed with both finer sand and coarser pebbles requires no reduction in fineness modulus provided the quantity of crushed stone is less than 30% of the total volume of the aggregate.

Mix	Size of Aggregate													
Cem.-Agg.	0-28	0-14	0-8	0-4	0-3*	0-⅜	0-½*	0-¾	0-1 in.*	0-1½	0-2.1*	0-3 in.	0-4½*	0-6 in.
1-12......	1.20	1.80	2.40	2.95	3.35	3.80	4.20	4.60	5.00	5.35	5.75	6.20	6.60	7.00
1-9.......	1.30	1.85	2.45	3.05	3.45	3.85	4.25	4.65	5.00	5.40	5.80	6.25	6.65	7.05
1-7.......	1.40	1.95	2.55	3.20	3.55	3.95	4.35	4.75	5.15	5.55	5.95	6.40	6.80	7.20
1-6.......	1.50	2.05	2.65	3.30	3.65	4.05	4.45	4.85	5.25	5.65	6.05	6.50	6.90	7.30
1-5.......	1.60	2.15	2.75	3.45	3.80	4.20	4.60	5.00	5.40	5.80	6.20	6.60	7.00	7.45
1-4.......	1.70	2.30	2.90	3.60	4.00	4.40	4.80	5.20	5.60	6.00	6.40	6.85	7.25	7.65
1-3.......	1.85	2.50	3.10	3.90	4.30	4.70	5.10	5.50	5.90	6.30	6.70	7.15	7.55	8.00
1-2.......	2.00	2.70	3.40	4.20	4.60	5.05	5.45	5.90	6.30	6.70	7.10	7.55	7.95	8.40
1-1.......	2.25	3.00	3.80	4.75	5.25	5.60	6.05	6.50	6.90	7.35	7.75	8.20	8.65	9.10

*Considered as "half-size" sieves; not used in computing fineness modulus.

values higher than given for relative consistency of 1·10 should seldom be considered in designing, since it is only in exceptional cases that a consistency drier than this can be satisfactorily placed. For wetter concrete much lower strengths must be considered.

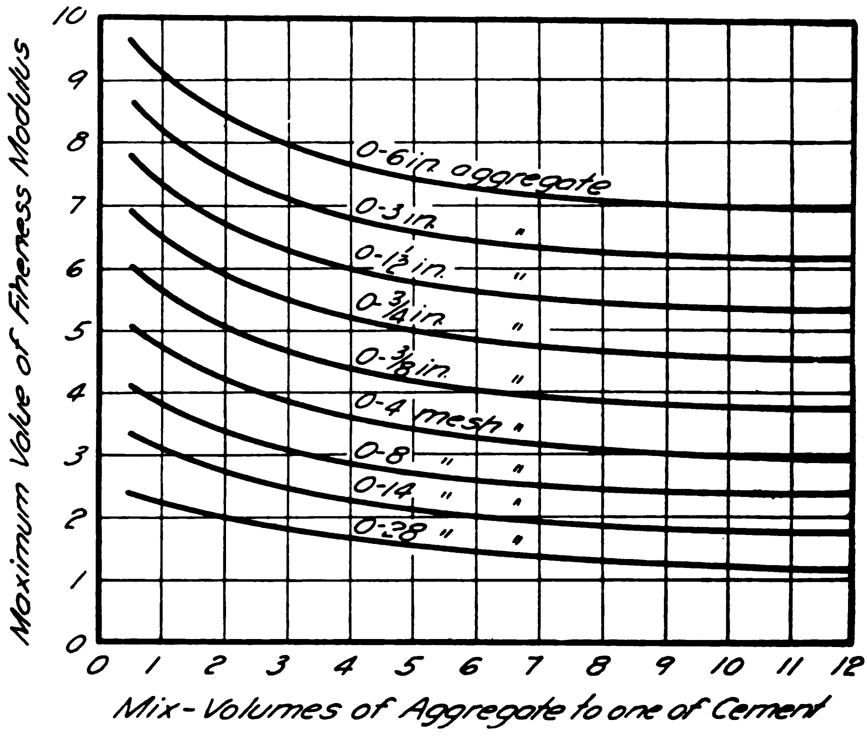
CALCULATION OF WATER REQUIRED FOR CONCRETE.

Because of the important influence of the quantity of water in the concrete it is desirable to have a sound basis for proportioning the water. The quantity of water necessary for given proportions and conditions may be determined by the following formula :—

$$x = R\left[\frac{3}{2}p + \left(\frac{\cdot 30}{1\cdot 26^{m}} + a - c\right)n\right] \quad . \quad (4)$$

where x = water required—ratio to volume of cement in batch (water-ratio).

R = Relative consistency of concrete, or "workability factor." Normal consistency (relative consistency = 1·00) requires the use of such a quantity of mixing water as will cause a slump of ½ to 1 in. in a freshly moulded 6-in. by 12-in. cylinder of about 1 : 4 mix upon withdrawing the form by a steady upward pull. A relative consistency of 1·10 requires the use of 10 per cent. more water, and under the above conditions will give a slump of about 5 in. to 6 in.



Graphical reproduction of Table 3. These curves are based on the requirements of sand and pebble aggregate. For crushed stone aggregate the values must be reduced as noted in the Table.

FIG. 5.—MAXIMUM PERMISSIBLE VALUES OF FINENESS MODULUS OF AGGREGATE.

p = Normal consistency of cement, ratio by weight.
m = Fineness modulus of aggregate (an exponent).
n = Volumes of mixed aggregate to one of cement.
a = Absorption of aggregate, ratio of water absorbed to volume of aggregate. (Determined after immersion in water for three hours. Average values for crushed limestone and pebbles may be assumed as 0·02; porous sandstones may reach 0·08; very light and porous aggregate may reach 0·25.)
c = Moisture contained in aggregate, ratio of water contained to volume of aggregate. (Assume as zero for room-dry aggregate.)

This formula takes account of all the factors which affect the quantity of water required in a concrete mixture. These factors may be classified as follows:—

(1) "Workability" factor, or the relative consistency of the concrete.—This is dictated by the kind of work being done; concrete must be more plastic (which generally means a wetter consistency) in reinforced concrete building construction than is necessary in mass work. The term (R) in the equation takes care of this factor. (R) may vary from, say, 0·90 for a dry concrete to 2·00 or higher for very wet mixes.

(2) Cement factor, which is made up of two parts: the quality of cement so far as normal consistency is concerned (p); the quantity of cement in the mix (n).

(3) The aggregate factor. This includes the three terms within the parenthesis in Equation (4). The first term, involving (m), takes account of the size and grading; the second (a) the absorption, and the third (c) the water contained in the aggregate.

In case admixtures of any kind are used, another term must be inserted in the equation. This relation has been fully worked out, but is not included in this report.

SIMPLIFIED WATER FORMULA.

While Equation (4) represents the true water relation, it is somewhat complicated by the fact that the fineness modulus (m) appears as an exponent. The equation

TABLE 4.—EXAMPLE OF INFLUENCE OF QUANTITY OF MIXING WATER ON THE STRENGTH OF CONCRETE.

Values calculated from the equation $S = \frac{A}{B^x} = \frac{14{,}000.}{8{\cdot}2^x}$

Where S = Compressive strength of concrete (lb. per sq. in.).
x = Water-ratio (an exponent).

A and B are constants whose values depend on quantity of cement and other conditions of the test. The values given for A and B are based on 28-day tests of 1 : 4 mix, pebble aggregate graded 0–1¼ in., fineness modulus 5·75.

The water-ratio is equivalent to 1 cu. ft. of water to 1 sack (1 cu. ft.) of cement.

The strength values are solely for comparative purposes in showing the influence of changing the water content.

Water in a 1-Bag Batch		Relative Consistency	Compressive Strength of Concrete at 28 Days	
Gallons	Water-Ratio (x)	Per Cent	Lb. per Sq. In. (S)	Relative Strength Per Cent
5.75	.77	100	2,770	100
6.0	.80	104	2,600	94
6.25	.84	109	2,400	87
6.5	.87	113	2,250	81
7.0	.94	122	1,950	70
7.5	1.01	131	1,670	60
8.0	1.07	139	1,470	53
9.0	1.21	157	1,100	40
10.0	1.34	174	830	30
12.0	1.60	208	480	17
15.0	2.00	260	200	7

TABLE 5.—QUANTITY OF MIXING WATER REQUIRED FOR CONCRETE.

Calculated by formula : $x = R\left[\frac{3}{2}p + \left(\frac{\cdot 30}{1\cdot 26^{m}} + a - c\right)n\right]$

Where x = Water required—ratio to volume of cement in batch (water-ratio).
R = Relative consistency, or "workability factor." Where $R = 1{\cdot}00$ the concrete is said to be of "normal consistency."
p = Normal consistency of cement by weight (assume $p = 0{\cdot}23$).
m = Fineness modulus of aggregate.
n = Volume of mixed aggregate to one volume of cement.
a = Absorption of aggregate, ratio of water absorbed to volume of aggregate.
c = Moisture in aggregate, ratio of water contained to volume of aggregate.
$(a - c)$ = Net absorption of aggregate by volume.

In this table $(a - c)$ is assumed as 0·02. In other words the net quantity of water taken by the aggregate is 2% by volume. This value may be used for ordinary limestones and pebbles. For crushed trap and granite it is somewhat high. It is too high in any case where the aggregate is saturated with water.

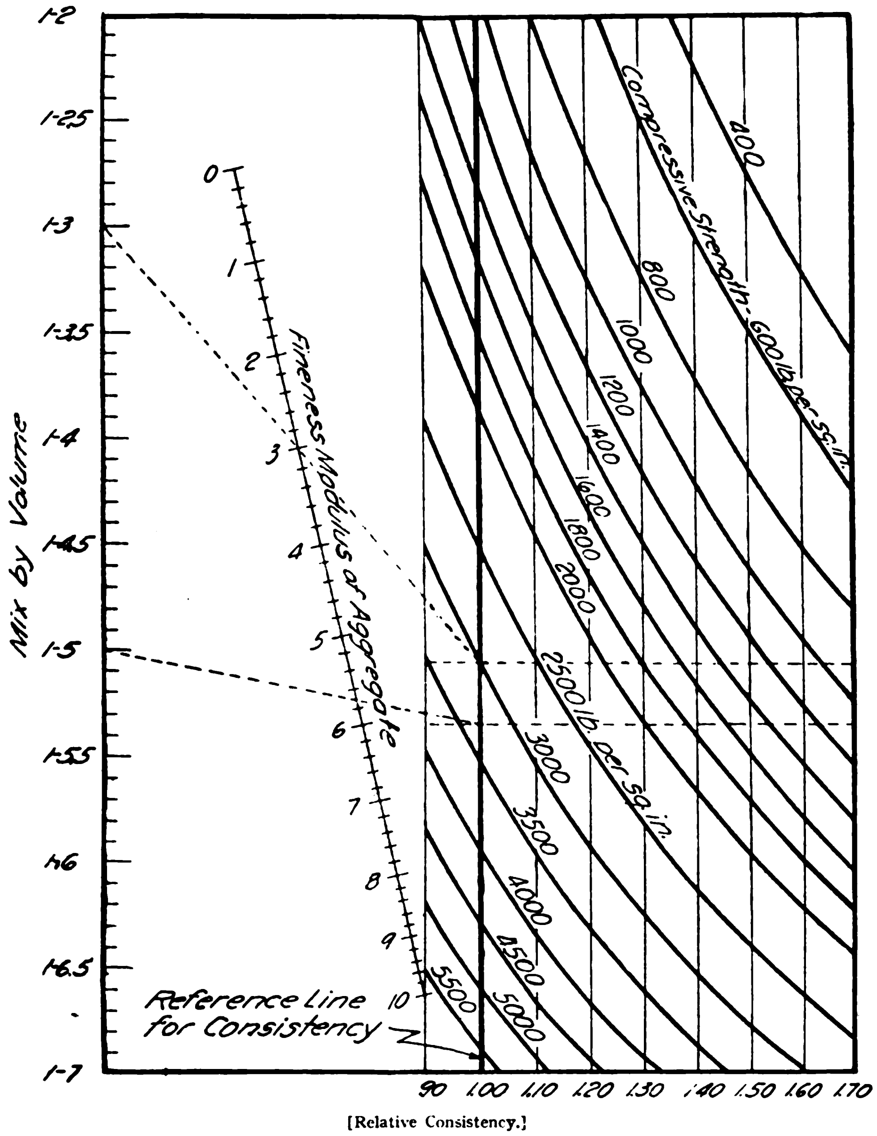
A relative consistency of 1·00 (normal consistency) requires the use of such a quantity of mixing water as will cause a slump of ½ to 1 in. in a freshly moulded 6 by 12-in. cylinder of about 1 : 4 mix upon withdrawing the form by a steady, upward pull. This consistency is somewhat dry for most concrete work, but can be used where light tamping is practicable.

A relative consistency of 1·10 (10% more water than required for normal consistency) represents about the driest concrete which can be satisfactorily used in concrete road construction. Under the conditions mentioned above, this consistency will give a slump of about 5 to 6 in.

A relative consistency of 1·25 represents about the wettest consistency which should be used in reinforced concrete building construction. Under the conditions mentioned above, this consistency will give a slump of about 8 to 9 in.

For mixes and fineness moduli, other than those given in the table, approximate values may be determined by interpolation. For specific cases use the formula.

Mix Cem.-Agg. by Volume	Gallons of Water per Sack of Cement Using Aggregates of Different Fineness Moduli											
	1.50	2.00	2.50	3.00	3.50	4.00	4.50	5.00	5.50	6.00	6.50	7.00
	Relative Consistency — (R) = 1.00											
1-12.......	23.5	21.4	19.5	17.8	16.4	15.2	13.9	12.9	12.0	11.1	10.4	9.8
1-9........	18.1	16.7	15.2	14.0	12.9	12.0	11.0	10.2	9.6	9.0	8.4	7.9
1-7........	14.7	13.5	12.3	11.4	10.6	9.9	9.1	8.6	8.0	7.6	7.2	6.7
1-6........	13.0	12.0	11.0	10.2	9.5	8.9	8.3	7.7	7.3	6.8	6.5	6.2
1-5........	11.2	10.4	9.5	8.9	8.3	7.8	7.3	6.9	6.4	6.1	5.8	5.5
1-4........	9.5	8.9	8.2	7.7	7.2	6.8	6.3	6.0	5.7	5.4	5.2	5.0
1-3........	7.8	7.2	6.7	6.3	6.0	5.7	5.4	5.1	4.9	4.6	4.5	4.3
1-2........	6.0	5.7	5.4	5.1	4.9	4.7	4.5	4.3	4.1	4.0	3.9	3.8
1-1........	4.3	4.1	3.9	3.8	3.7	3.6	3.5	3.4	3.3	3.2	3.2	3.1
	Relative Consistency — (R) = 1.10											
1-12.......	25.8	23.6	21.4	19.6	18.1	16.7	15.3	14.2	13.2	12.2	11.4	10.8
1-9........	19.9	18.4	16.7	15.4	14.2	13.2	12.1	11.2	10.6	9.9	9.2	8.7
1-7........	16.2	14.9	13.5	12.5	11.7	10.9	10.0	9.5	8.8	8.4	7.9	7.4
1-6........	14.3	13.2	12.1	11.2	10.5	9.8	9.1	8.5	8.0	7.5	7.2	6.8
1-5........	12.3	11.4	10.5	9.8	9.1	8.6	8.0	7.6	7.0	6.7	6.4	6.1
1-4........	10.5	9.8	9.0	8.5	7.9	7.5	6.9	6.6	6.3	5.9	5.7	5.5
1-3........	8.6	7.9	7.4	6.9	6.6	6.3	5.9	5.6	5.4	5.1	5.0	4.7
1-2........	6.6	6.3	5.9	5.6	5.4	5.2	5.0	4.7	4.5	4.4	4.3	4.2
1-1........	4.7	4.5	4.3	4.2	4.1	4.0	3.9	3.7	3.6	3.5	3.5	3.4
	Relative Consistency — (R) = 1.25											
1-12.......	29.4	26.8	24.4	22.2	20.5	19.0	17.4	16.1	15.0	13.9	13.0	12.3
1-9........	22.6	20.9	19.0	17.5	16.1	15.0	13.8	12.7	12.0	11.2	10.5	9.9
1-7........	18.4	16.9	15.4	14.3	13.2	12.4	11.4	10.7	10.0	9.5	9.0	8.4
1-6........	16.3	15.0	13.8	12.8	11.9	11.1	10.4	9.6	9.1	8.5	8.1	7.7
1-5........	14.0	13.0	11.9	11.1	10.4	9.8	9.1	8.6	8.0	7.6	7.2	6.9
1-4........	11.9	11.1	10.2	9.6	9.0	8.5	7.9	7.5	7.1	6.8	6.5	6.2
1-3........	9.8	9.0	8.4	7.9	7.5	7.1	6.8	6.4	6.1	5.8	5.6	5.4
1-2........	7.5	7.1	6.8	6.4	6.1	5.9	5.6	5.4	5.1	5.0	4.9	4.8
1-1........	5.4	5.1	4.9	4.8	4.6	4.5	4.4	4.3	4.1	4.0	4.0	3.9



This chart is based on compression tests of 6-in. by 12-in. cylinders; age 28 days; stored in deep sand. The cement used gave compressive strengths in 1–3 standard mortar as follows: 7 days, 1,900 lb. per sq. in.; 28 days, 3,200 lb. per sq. in.; 3 months 4,200 lb. per sq. in.; 1 year, 4,300 lb. per sq. in.

Fig. 6.—Diagram for the Design of Concrete Mixtures.

can be expressed in a simpler form as follows :—

$$x = R\left[\frac{3}{2}p + \left(0{\cdot}22 - \frac{m}{42} + a - c\right) n\right] \quad . (5)$$

This equation gives values for ordinary ranges of mix and grading of aggregate which are sensibly the same as given by Equation (4).

MAXIMUM PERMISSIBLE VALUES OF FINENESS MODULUS OF AGGREGATES.

Since a maximum practicable value of fineness modulus is found for each size of aggregate and mix, it is necessary to place certain limits on the value which may be used for proportioning materials for concrete mixes. Table 3 gives limits which will be found practicable. Subsequent experience may dictate certain modifications in the details. The purpose of Table 3 is to avoid the attempt to secure an aggregate grading which is too coarse for its maximum size and for the amount of cement used. It is also useful in prohibiting attempts to use sands which are too coarse for best results in concrete mixtures.

The curves in *Fig.* 5 are plotted directly from the values given for the standard sieves in Table 3.

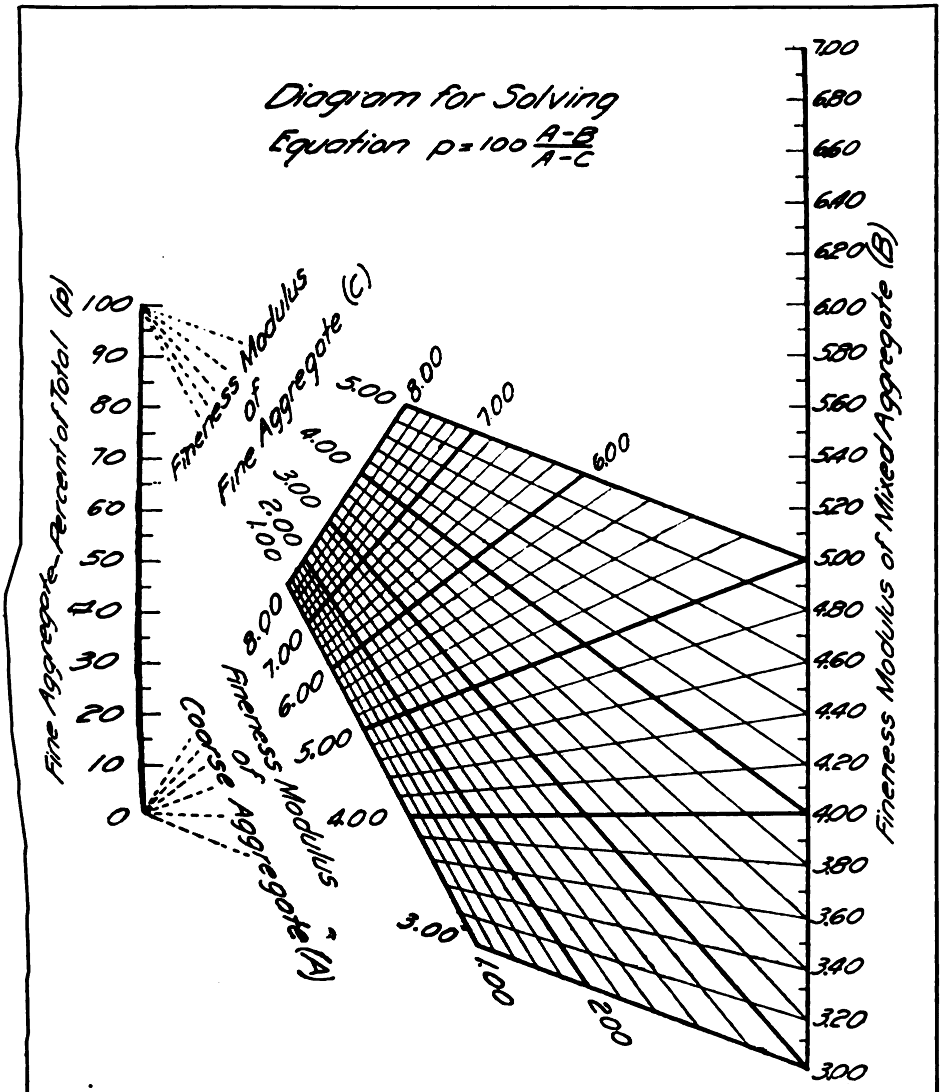
CHART FOR DESIGN OF CONCRETE MIXES

Fig. 6 is a nomographic chart for the design of concrete mixes. This chart takes account of the following four factors :—(1) The mix (cement content) ; (2) the relative consistency ; (3) the grading of aggregate (fineness modulus) ; (4) the compressive strength of concrete. Given any three of these factors the chart enables one to solve for the fourth. This chart is based on the results of tests. For practical application these values must generally be reduced by certain factors, which will depend on the judgment of the designer. In order to furnish some basis for comparison, compression tests of 1 : 3 standard sand mortars from the cement used in these tests are given.

Consider the case of concrete for road construction. This is generally specified as a 1 : 1½ : 3 or a 1 : 2 : 3 mix, with aggregate graded up to 1½ in. These mixes are about the same as what have been termed a 1 : 4 mix, the exact equivalent depending on the particular size and grading of the fine and coarse aggregate. Assume that gravel aggregate will be used, graded to 1½ in. Table 3 shows that a fineness modulus as high as 6·00 – ·25 = 5·75 may be used. Knowing the sieve analysis and fineness modulus of both sizes of aggregate, apply the formula or *Fig.* 7 to determine the proportions of each aggregate which must be mixed to secure this value. Assume that the concrete will be mixed to a relative consistency of 1·10, which is of such plasticity as will give a slump of 5 to 6 in. in the test described above. Place a straight-edge in *Fig.* 6 on mix 1 : 4 and fineness modulus 5·75, and mark the point where it crosses the reference line for consistency ; from this point project the line horizontally (as indicated in other examples) to relative consistency 1·10. It will be seen that this gives a compressive strength of 3,400 lb. per sq. in. at 28 days.

The effect of using other mixes, gradings, or consistencies on the strength can be seen at once from the diagram. For instance, if the water were increased to a relative consistency of 1·25 (not nearly so wet as is frequently seen in road work) the strength will be reduced to 2,700 lb. per sq. in.—a reduction of over 20 per cent. If the mix were changed to 1 : 4½ and other factors the same as in the first example, the strength would be 3,200 lb. per sq. in. We should have to change the mix to as lean as 1 : 5½ in order to secure the same reduction in strength as was found above for a change from 1·10 to 1·25 consistency. By using the wetter of the two consistencies we secure concrete of the same strength as if we had used one-third less cement and the drier mix. In other words, increasing the mixing water 13 per cent. causes the same reduction in strength as if we should omit 33 per cent. of the cement. This example shows the reason for emphasising the importance of proper control of mixing water in concrete.

This chart enables us to answer such questions as the following : Which is the stronger, a 1 : 3 mortar or a 1 : 5 concrete mixture ? Assuming that concrete of the same plasticity is used, the relative strengths will depend, of course, on the grading of the aggregates and the mix. In one case we have assumed 1 : 3 mix with fineness modulus equal to 3·00.



Based on equation $p = 100\frac{A - B}{A_x\ C_x}$.

Where p = percentage of fine aggregate in total mixture.
A = fineness modulus of coarse aggregate.
B = fineness modulus of total aggregate.
C = fineness modulus of fine aggregate.

FIG. 7.—DIAGRAM FOR DETERMINING QUANTITY OF SAND REQUIRED IN CONCRETE MIXES.

This will give a strength for normal consistency of 3,000 lb. per sq. in. The 1 : 5 mix (fineness modulus 5·70) gives a strength for normal consistency of about 3,300 lb. per sq. in. The strengths for other consistencies can be found by reading horizontally across the chart as indicated by the dotted lines.

Unfortunately, we now have no proper basis for absolute values for strength of concrete. This, of course, makes it necessary to refer to particular tests as in *Fig.* 6. This condition emphasises the importance of working out a test of cement which will give at once the concrete strength for given materials, mixes, etc. With the present method of testing cement it is possible to do more than make a rough guess as to the strength of concrete from the results of briquet tests.

QUANTITY OF WATER REQUIRED FOR CONCRETE.

Formulas 4 and 5 show the elements which make up the water-requirements of a concrete mix. Table 5 gives the quantity of water required for certain mixes and values of fineness modulus. Quantities are given in terms of gallons per sack of cement. In this table the net absorption (that is, the quantity of water taken up by the aggregate in addition to that already contained) is assumed as 0·02 (2 per cent. by volume). This table is of interest when we consider that it has been found that a given water-ratio corresponds to constant concrete strength regardless of the combination of mix, consistency or grading of aggregate which may be used, so long as we have a workable concrete.

FURTHER DISCUSSION OF CONCRETE MIXES.

The importance of the water-ratio on the strength of concrete will be shown in the following considerations: One pint more water than necessary to produce a plastic concrete reduces the strength to the same extent as if we should omit 2 to 3 lb. of cement from a 1-bag batch. Our studies give an entirely new conception of the function performed by the various constituent materials. The use of a coarse, well-graded aggregate results in no gain in strength unless we take advantage of the fact that the amount of water necessary to produce a plastic mix can thus be reduced. In a similar way we may say that the use of more cement in a batch does not produce any beneficial effect except from the fact that a plastic workable mix can be produced with a lower water-ratio. The reason a rich mixture gives a higher strength than a lean one is not that more cement is used, but because the concrete can be mixed (and usually is mixed) with a water-ratio which is relatively lower for the richer mixtures than for the lean ones. If advantage is not taken of the fact that in a rich mix relatively less water can be used, no benefit will be gained as compared with a leaner mix. In all this discussion the quantity of water is compared with the quantity of cement in the batch (cu. ft. of water to 1 sack of cement) and not to the weight of dry materials or of the concrete as is generally done.

The mere use of richer mixes has encouraged a feeling of security, whereas in many instances nothing more has been accomplished than wasting a large quantity of cement, due to the use of an excess of mixing water. The universal acceptance of this false theory of concrete has exerted a most pernicious influence on the proper use of concrete materials and has proved to be an almost insurmountable barrier in the way of progress in the development of sound principles of concrete proportioning and construction. Rich mixes and well-graded aggregates are just as essential as ever, but we now have a proper appreciation of the true function of the constituent materials in concrete, and a more thorough understanding of the injurious effect of too much water. Rich mixes and well-graded aggregates are after all only a means to an end; that is, to produce a plastic workable concrete with a minimum quantity of water as compared with the cement used. Workability of concrete mixes is of fundamental significance. This factor is the only limitation which prevents the reduction of cement and water in the batch to much lower limits than are now practicable.

The above considerations show that the water content is the most important element of a concrete mix, in that small variations in the water cause a much wider change in the strength than similar variations in the cement content or the size or grading of the aggregate. This shows the absurdity of the present practice in specifying definite gradings for aggregates and carefully proportioning the cement, then guessing at the water. It would be more correct carefully to measure the water and guess at the cement in the batch.

The grading of the aggregate may vary over a wide range without producing any effect on concrete strength, so long as the cement and water remain unchanged. The consistency of the concrete will be changed, but this will not affect the concrete strength if all mixes are plastic. The possibility of improving the strength of concrete by better grading of aggregates is small as compared with the advantages which may be reaped from using as dry a mix as can be properly placed. Table 4 shows the effect of water on the strength of concrete.

It is impracticable to lay down a general rule for the quantity of water which should be used in a concrete mix, since it was seen in the water formulæ that the total water is governed by a large number of different factors. However, it is only water which goes to the cement (that is, exclusive of absorbed water) which affects the concrete strength. The failure to recognise this fact has led to many erroneous conclusions from tests carried out with the object of determining the relative merits of different aggregates.

Table 5 gives the quantity of water required for plastic mixes for certain assumed conditions of normal consistency of cement, absorption of aggregate, and relative consistency. Water is expressed in terms of gallons per sack of cement. In using this table the dependence of the value of fineness modulus which may be used on the size of aggregate and the mix, referred to in Table 3, should not be overlooked.

Without regard to the actual quantity of mixing water, the following rule is a safe one to follow: *Use the smallest quantity of mixing water that will produce a plastic or workable concrete.* The importance of any method of mixing, handling, placing and finishing concrete which will enable the builder to reduce the water content of the concrete to a minimum is at once apparent.

Decoration of Concrete Surfaces.

CEMENT and fine concrete surfaces will, says the *Building News*, take embossed decoration quite well. This can be applied by using the chisel or by employing stamps or rollers of wood or metal having raised or sunken designs, much in the same fashion as in old pargetting. These stamps and rollers must be constantly dipped in water while in service, for their faces and sides must be thoroughly wet whenever they come into contact with the cement. Sunk patterns have their merits; not only are they effective artistically, but in many positions they have a mechanical advantage over raised ornament—for instance, wherever the ornament is less than 6 ft. above ground level, whether in or out of doors. A style worthy of note, because looking particularly well when applied to cement or concrete, is the combination of sinking and embossing which we find in cavo-relief. This is a method much used by the ancient Egyptians, sometimes seen in Celtic work, and associated with gem carving. The design, whether it be figure or floral work or conventional pattern, is carved so as to give more or less relief and rounding, but no part of the design is on a higher level than the general surface. That is to say, the design is traced on the surface, then panels are traced round the designs, and the material within the panel cut away to the depth desired, taking care not to injure the pattern. When the superfluous material has been removed it is possible to attack the pattern, carrying out the design by the use of hammer and chisel or other suitable instruments, and giving it all the prominence of detail and roundness necessary. It is possible to execute elaborate carving in this way, also very striking lettering, but also quite simple patterns can be applied with the help of templates and stamps. Cavo-relief is much used for monuments. It is also well adapted for panels on large and otherwise uninteresting wall surfaces, for small rectangular or fan-shaped panels over door and window openings, and also for banding as a frieze or a string-course.

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Action of Sub-Soil Water on Concrete.

Herr H. Lührig has published a warning respecting the effect on concrete of sub-soil water containing sulphate of iron or of manganese. He has found that both these substances behave like free sulphuric acid. Water in contact with iron sulphide (pyrites) and air may form iron sulphate, and so have a similar effect. Curiously enough, he has not found other sulphates to have any serious action on concrete.—*Wasser u. Gas.*

Test on the Wembley Stadium.

Messrs. David Kirkaldy & Son have carried out a series of tests of the new Empire Stadium at Wembley Park. The tests were made chiefly by the application of some thousands of sand bags, and by the marching of men over the structure. On an area intended to accommodate 2,000 spectators, a load of sand equal to the weight of 4,000 people was applied, and no signs of weakness were observed. Staircases were subjected to a similar intensity of loading with equally satisfactory results. A body of 1,200 men marched about the Stadium in close formation; they stood and sat in unison, marked time in step, and swayed and surged upon the barriers, but no dangerous strains were caused by these operations.

The Stadium is 896 ft. long by 695 ft. wide, and will accommodate 125,000 spectators. In the construction of the structure the following materials have been used: 1,400 tons of structural steelwork; half a million rivets; 600 tons of steel rods for reinforcing the concrete—the total length of these rods is about 500 miles, and the total number of the rods 250,000; 25,000 tons of concrete (using 3,500 tons of cement); 40 miles of terracing; and 14 miles of concrete beams to form the terracing in the stands.

Reinforced Concrete Road in Flintshire.

A meeting of the members of the North Wales Branch of the Surveyors' Institution was held at Rhyl, recently, when the party was conducted over the new road in course of construction between Rhyl and Gronant, through Prestatyn, by Mr. R. G. Whitley, A.M.Inst.C.E., County Surveyor of Flintshire, and his assistant, Mr. Cathrall, P.A.S.I. The new road is a continuation of the existing coast road, and avoids the many turns and dangerous gradients of the existing road. The total length of the road so far under construction is about six miles, half of which is of reinforced concrete. The estimated cost for the six miles is £36,000, of which the Ministry of Transport is bearing half the cost. The foundation is extremely bad, being over spongy marsh land, and the road had to be raised for 2¼ miles so as to be above high-tide level; and as no filling material was available the road was raised by means of sand. Mr. Whitley explained that the road was 20 ft. wide and 50 ft. between fences, with provision for widening, and was formed so that the top surface had a cross-fall of 1 in 50, the thickness of the concrete road in the centre being 8½ in. The reinforcement was laid, and the concreting done, in 13-ft. alternate bay sections in the direction of the length of the road. Walker-Weston double-layer interlocked reinforcement is being used throughout the work.

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	6
Clean Shingle, ¾ in. mesh	,,	14	6
,, ,, ½ in. mesh	,,	15	6
Best British Portland Cement	per ton 58*s.* to	63	0
"Ferrocrete" Portland Cement	10*s.* per ton extra		

Boarding for Shuttering—

		Sawn.		*Wrot.*	
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

Sawn Timber for Struts and Supports—

3 in. by 4 in.	from £23 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £24 ,, ,,

Mild Steel Rods for Reinforcement—

		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	14	6
9/16 in. to ½ in. Rounds	,,	14	6
⅜ in. Rounds	,,	15	6
¼ in. Rounds	,,	17	6

MATERIAL AND LABOUR INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

Portland Cement Concrete well mixed with Clean Water in proportion.

1 : 2 : 4—			s.	d.
Do.	do. in foundation	per yard cube	45	0
Do.	do. in columns	,, ,,	50	0
Do.	do. in beams	,, ,,	50	0
Do.	do. in floor slabs 4 in. thick	per yard super	5	3
Do.	do. in floor slabs 6 in. thick	,, ,,	7	9
Do.	do. in floor slabs 9 in. thick	,, ,,	11	3
Do.	do. in walls 6 in. thick	,, ,,	8	0

(*Add for hoisting* 3*s.* 6*d. per yard cube above ground-floor level.*)

Steel Reinforcement, including Cutting, Bending and putting into Position and securing with Stout Binding Wire—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	27	6
,, 9/16 in. to ½ in.	,,	25	6
,, ⅝ in. to 2⅞ in.	,,	24	6

Shuttering—

		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	60	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	50	0
Do. do. in small quantities	per ft. super	0	10
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	11
Do. do. as last in narrow widths	,, ,,	1	1
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	1
Do. do. as last in narrow widths	,, ,,	1	3
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

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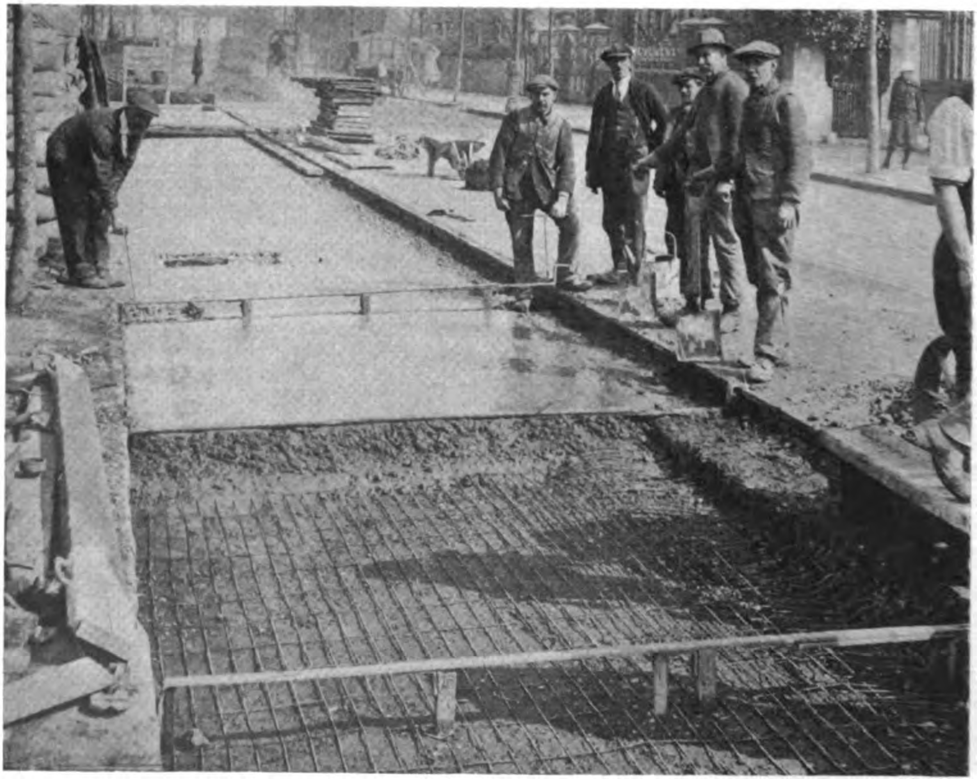
A NEW SCREED.

THE patent screed (the " Waithman ") illustrated below has been put on the market by Messrs. Brown & Tawse, Ltd., of 3 London Wall Buildings, London, E.C.2. The chief advantage of this useful tool is that it enables a concrete road or road foundation to be laid continuously without in any way interfering with the placing of the concrete. It is thus possible to place the concrete quickly so that it forms a homogeneous mass both below and above the reinforcement without the concrete below the reinforcement having time to set. The screed has been used on several road works, and its advantages have been testified to by Mr. Alfred Dryland, M.Inst.C.E., County Engineer of Middlesex, who, after referring to the possibility it presents of laying concrete roads quickly and advantageously, has written: " Such defects as arise in concrete under wood paving are, in my opinion, largely due to the process of applying thin floating coats, which are liable to separate from the body of the concrete and be crushed. The process of forming concrete screeds, which is a necessary part of this method, introduces coves which are in themselves undesirable, whilst the subsequent floating, followed at a still later stage by the making good after the removal of wood laths, involves three operations and three different periods of setting, tending to inequality in the concrete. The use of the Waithman's screeds eliminates entirely two of these operations and a complete foundation is provided equal in quality throughout and all setting together with the reinforcement properly placed and bonded with the concrete. I consider the screed a most useful invention and shall commend it to the attention of my colleagues." The tool is extremely simple to handle and portable, and is easily fixed and withdrawn.



THE "WAITHMAN" SCREED.

REINFORCEMENT FOR CONCRETE PAVEMENTS.

In a recent issue of *Engineering News-Record*, Mr. H. Eltinge Breed, Consulting Engineer, New York City, discusses very fully the use of steel reinforcement for concrete pavements. In the writer's opinion reinforcement tends to lessen the cracking in the pavement. He states that in 1919 more miles of concrete were laid in the United States than of any other type of durable pavement. The growth of the popularity of concrete as indicated by the laying of 941,659 sq. yds. in 1910 and 52,000,000 sq. yds, in 1919, has been little short of phenomenal. In response to a request for suggestions for an ideal section of the Lincoln Highway sent out by the Lincoln Highway Association, twenty-four of the forty replies which indicated a preference for a certain type of pavement specified concrete and nine of these suggested steel reinforcement. It is obvious that in many localities the concrete road is the cheapest of the durable types of pavement, that its cost of maintenance is low, and that people like it because it can be depended upon for service 365 days a year.

A. T. Goldbeck, engineer of tests, Bureau of Public Roads, reports some interesting experiments to determine the force of impact upon uneven surfaces. He says, in the proceedings of the American Concrete Institute :

" We have conducted a number of tests, using trucks of different size, in order to determine something of the amount of impact exerted on concrete roads, and it is rather surprising to find out just how much this impact can be. For instance, say you have a heavy truck, a three-ton army truck, which is very often a five-ton truck, and that truck has a total weight on one rear wheel of 7,500 lb. when the truck is loaded with five tons. When one rear wheel of that truck falls through a height of ½ in., the impact on the road may be as high as 20,000 lb. ; that is, the impact pressure produced may be as high as 20,000 lb., almost three times as much as the static road pressure. If the truck falls through a height of 3 in., which would, of course, mean an exceedingly bad road, you may get an impact pressure as high as 43,000 lb.

To build roads to withstand such impact as this last would be financially ruinous. What we can and must do is to build roads of such smoothness that there will be no intensification of the impact pressure.

After dealing with the question of road stresses, and the question of the two-layer reinforcement, the author throws out a few suggestions for future reinforcement. Those suggestions are briefly summarised, as follows :—

1. One of the great dangers in using reinforcing steel is that road builders, given an additional factor of safety, may be tempted to use a leaner mix, poorer materials and poorer workmanship. Such construction would only defeat its own purpose.

2. On account of the fact that transverse cracks are not as dangerous as longitudinal cracks, and as the latter are the ones to be eliminated if possible on account of the destructive effect of traffic running along them, it would seem wise to take a metal that has a ratio of from one, lengthwise, to from three and one-half to five, across the pavement. In city work, however, where transverse cracks are more detrimental, the ratio should be one, lengthwise, to from one to two and one-half, crosswise, depending on the length of the slab.

3. With the increased use of steel, slabs can be lengthened without the danger of transverse cracks.

Conclusions.—The analogy between the use of steel reinforcing for structural work and highways will not hold, but the writer believes that reinforcement will be a factor in helping us build better roads.

It is upon the observations made, that the writer bases his statement about the two-fold function of steel reinforcing : It preserves the integrity of the pavement, and prolongs its life.

All Readers of this Journal

are cordially invited
to pay a visit to the

Permanent Exhibition

of

Concrete Products

at

143 Grosvenor Road, S.W.1
(Near Vauxhall Bridge)
OPEN DAILY (*Saturdays excepted*)
10.0 TO 4.0. - - ADMISSION FREE

The collection includes examples of Concrete in the following interests :—

ARCHITECTS AND BUILDERS, ESTATE, FARM AND HOME, MUNICIPAL AUTHORITIES, RAILWAYS AND OTHERS.

The Exhibition has been arranged by the—

CONCRETE UTILITIES BUREAU
35 GREAT ST. HELENS - - - LONDON, E.C.3.

"This Exhibition shows in a comprehensive manner the multitudinous uses of Concrete . . . . and should be visited by all interested in the material."—*The Builder.*

A Presentation.

An interesting presentation was recently made to Mr. C. C. Hancock, Resident Engineer for the past two years to the Durham County Council on the East Coast Road, who has now resigned that appointment to enter the commercial side of municipal engineering, when he was presented by the staff with a silver rose bowl. The Works Committee of the Durham County Council has also recorded its appreciation of the efficient services of Mr. Hancock during the two years he had held the appointment. Mr. Hancock has now joined the staff of The Expanded Metal Company, Ltd., of London and West Hartlepool, and, under the existing management, will take charge of the Roads and Public Works Department of that firm.

Reinforced Concrete Road at King George V Sanatorium.

We omitted to state in our last issue that the reinforced concrete road at King George V Sanatorium, Godalming, illustrated in our Frontispiece, was reinforced by B.R.C. Fabric, and that we were indebted to the B.R.C. Company for the loan of the photograph.

PROSPECTIVE NEW CONCRETE WORK.

Bootle.—*Road Works.*—The T.C. proposes borrowing £27,259 for widening and reconstructing certain roads.

Boscombe.—*Landing Stage.*—The Bournemouth Corporation is considering a proposal to construct a landing stage at Boscombe at an estimated cost of £25,000.

Bournemouth. —*Defence Works.* — The Ministry of Health has held an inquiry into an application by the T.C. for sanction to borrow £53,500 for Cliff protection works at Boscombe.

Carlton.—*Bridge, etc.*—The West Riding C.C. proposes to spend £38,300 on the construction of a new road and bridge over the River Aire at Carlton.

Castletown. *Road.*—The Sunderland R.D.C. has decided to construct a new road from Castletown to North Hylton. The estimated cost of a 30 ft. road is £7,129.

Cork.—*Dry Dock.*—The Harbour Commissioners are considering a proposal to construct a dry dock at an estimated cost of £13,000.

East Cowes.—*Esplanade.*—The U.D.C. has decided to proceed with the extension of the Esplanade at an estimated cost of £15,000.

Edinburgh.—*New Road.*—It is stated the Ministry of Transport has approved the construction of a new main road between Edinburgh and Glasgow. The total estimated cost is nearly £2,000,000.

Frinton. — *Sea Defence Works.* — The U.D.C. has adopted a scheme for the construction of a concrete revetment with reinforced concrete piles at a cost of £520, for the protection of the cliff between the slipway and Battery Point.

Hertford.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the T.C. for sanction to borrow money for sewage disposal works.

Hounslow.—*Road.*—The Middlesex C.C. has passed for immediate construction a new road between Hounslow and Harrow. The new road which will be 3½ miles long and 60 feet wide, will be made of concrete with an asphalt topping. The work is estimated to cost £160,555.

Hull.—*Road Work.*—A scheme for the construction of ring roads in the neighbourhood of Dousom Lane to the Garden Village projected by the Works Committee at an estimated cost of £22,265 has been approved by the Corporation Finance Committee.

Ilkley.—*Sewage Works.*—The U.D.C. has received sanction to borrow £2,200 for extensions to the sewage disposal works.

Kilwinning.—*Road Works.*—The T.C. has decided to proceed with the reconstruction of the principal street in the town at an estimated cost of £26,000.

Melton Mowbray. — *Reservoir.* — The U.D.C. has decided to construct a reservoir of 1,000,000 gallons capacity.

Morecambe.—*Ring Road.*—The T.C. has decided to apply to the Ministry of Health for sanction to borrow £39,000 towards the cost of constructing a road from the west end of the town. The total cost will be £78,000.

Newton Abbot.—*Station.*—The G.W.R. Co. has decided to reconstruct the station at a cost of about a quarter of a million pounds.

Poole.—*Bridge.*—The T.C. has decided to construct a new ferro-concrete bridge with a width of 28 ft., at Hamworthy. The estimated cost is £40,000.

Preston.—*Retaining Wall.*—The T.C. proposes to construct a retaining wall on the south side of the power station site at a cost of £2,162.

Ramsgate.—*Pier.*—It is proposed to erect a new structure in place of the Marine Pier, which was burnt down some years ago.

Saltcoats.—*Sea Wall.*—The T.C. has agreed to proceed with the construction of a sea wall at an estimated cost of £15,400.

Seaton Carew.—*Sea Wall.*—The West Hartlepool T.C. is considering a scheme for the widening of the front at Seaton Carew by the erection of a sea wall 1,600 ft. in length, and a distance of 90 ft. from the present wall, and the provision of a bathing pool, etc., at an estimated cost of £41,340.

SHEFFIELD. — *Wall.* — The Corporation Finance Committee has approved an expenditure of £3,224 on the construction of a retaining wall in Horninglow Road.

SHREWSBURY.—*Bridge.*—The T.C. has decided to proceed at once with the widening of the English Bridge across the River Severn at a cost of £70,000.

SOUTHGATE (Middlesex).—*Swimming Pool.* —The U.D.C. has decided to construct a swimming pool on a site near Powys-lane and Bowes-road.

TENBY.—*Road.*—The T.C. has received the sanction of the Ministry of Health to construct a new road from the end of the esplanade to the South Sands.

TIVERTON.—*Open-air Bath.*—The T.C. has requested the Property Committee to prepare a scheme for an open-air bath on the Loughborough site at a total cost of £1,500.

WALTHAMSTOW.—*Road Work.*—The Essex C.C. has approved the scheme of the Walthamstow Council to reconstruct a portion of Forest Road at an estimated cost of £20,057.

TENDERS ACCEPTED.

BARRY.—*Shelter, etc.*—The Barry U.D.C. has accepted the tender of Messrs. Vickery, Bros., Barry, at £9,576 for the erection and construction of shelter and shops in Hennebique ferro-concrete construction.

BELFAST.—*Reservoir.*—The Belfast Water Commissioners have accepted the tender of Messrs. Pearson & Co., Ltd., at £1,028,250 for the construction of a reservoir in the Silent Valley, Mourne Mountains, Co. Down.

PORTSMOUTH. *Quay Widening.*—The Corporation has accepted the tender of Messrs. W. Arrol & Co., 82 Victoria Street, S.W.1, at £3,978 4*s*. 3*d*. for the widening of a portion of the Quay in the Outer Camber in reinforced concrete construction, including maintenance for 12 months.

ROCHDALE.—*Humus Tanks.*—The Paving, etc., Committee has accepted the tender of Mr. A. E. Farr, 8–10 Great St. Helens, London, E.C.3, for the construction of two humus tanks in ferro-concrete in connection with the extension of the sewage disposal works at Rock Mills, Rochdale.

ROCHDALE. *Paving.*—The Paving, etc., Committee has accepted the tender of Mr. S. Kearsley, 32 Syke Road, Rochdale, for the concrete paving work required in Bridgefold Road.

SALTCOATS (Scotland).—*Sea Wall.*—The T.C. has accepted the tender of Mr. G. Porter, 432A Main Street, Bellshill, at £1,646 4*s*., for the construction of a concrete sea wall.

SHEFFIELD.—*Concrete Linings.*—The T.C. has accepted the tender of Messrs. Wellerman Bros., at £298 10*s*. for reinforced concrete linings to steel bunkers at Neepsend Works.

TRADE NOTICES.

A series of leaflets has been issued by the British Steel Piling Co., of Dock House, Billiter Street, E.C.3, describing their McKiernan-Terry pile-driving hammers, "Zenith" friction winch, and steel sheet piling.

Mr. Gilbert Ness, M.I.Struct.E., Representative Engineer of the Indented Bar and Concrete Engineering Co., Ltd., for the Yorkshire and East Midlands district, has removed to "Stonelea," Harlow Hill, Otley Road, Harrogate.

A pamphlet describing their 6-ton end-tipping wagon, gully-emptier, and street watering tank has been issued by Messrs. Mann's Patent Steam Cart and Wagon Co., Ltd., of Hunslet, Leeds. The ease with which the wagon may be changed from one to either of its other uses is a distinctive feature, and should specially appeal to local authorities whose area does not justify the purchase of separate vehicles for each purpose.

Messrs. Edward G. Herbert, Ltd., of Atlas Works, Chapel Street, Levenshulme, Manchester, are about to place on the market a new instrument for testing the hardness of various substances, which is specially applicable to hardened steels. The apparatus, weighing nearly 9 pounds, consists of a pendulum four-thousandths of an inch long, which is balanced on a ruby or steel ball one millimetre in diameter. When placed on any substance and set swinging by a touch with a feather, it indicates the hardness of the surface on which it rests by the time of swing, which is read with a stop watch. Another test with the same instrument depends on a different principle and measures another kind of hardness The pendulum is allowed to swing from a tilted position and the hardness is shown by the position, at the end of the swing, of a bubble in a curved tube against a graduated scale. The apparatus, we are informed, dispenses with all microscopic measurements and all difficult readings, and, as it does not depend on impact, the readings are in no way affected by the mass or inertia of the specimen. It is portable and quick in action, a test only occupying a few seconds. Heavy loads are not used and the test may be applied to thin and fragile articles without risk of breakage or defacement of finished surfaces.

JUN 22 1923

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

JUNE 1923 VOL. XVIII. No. 6

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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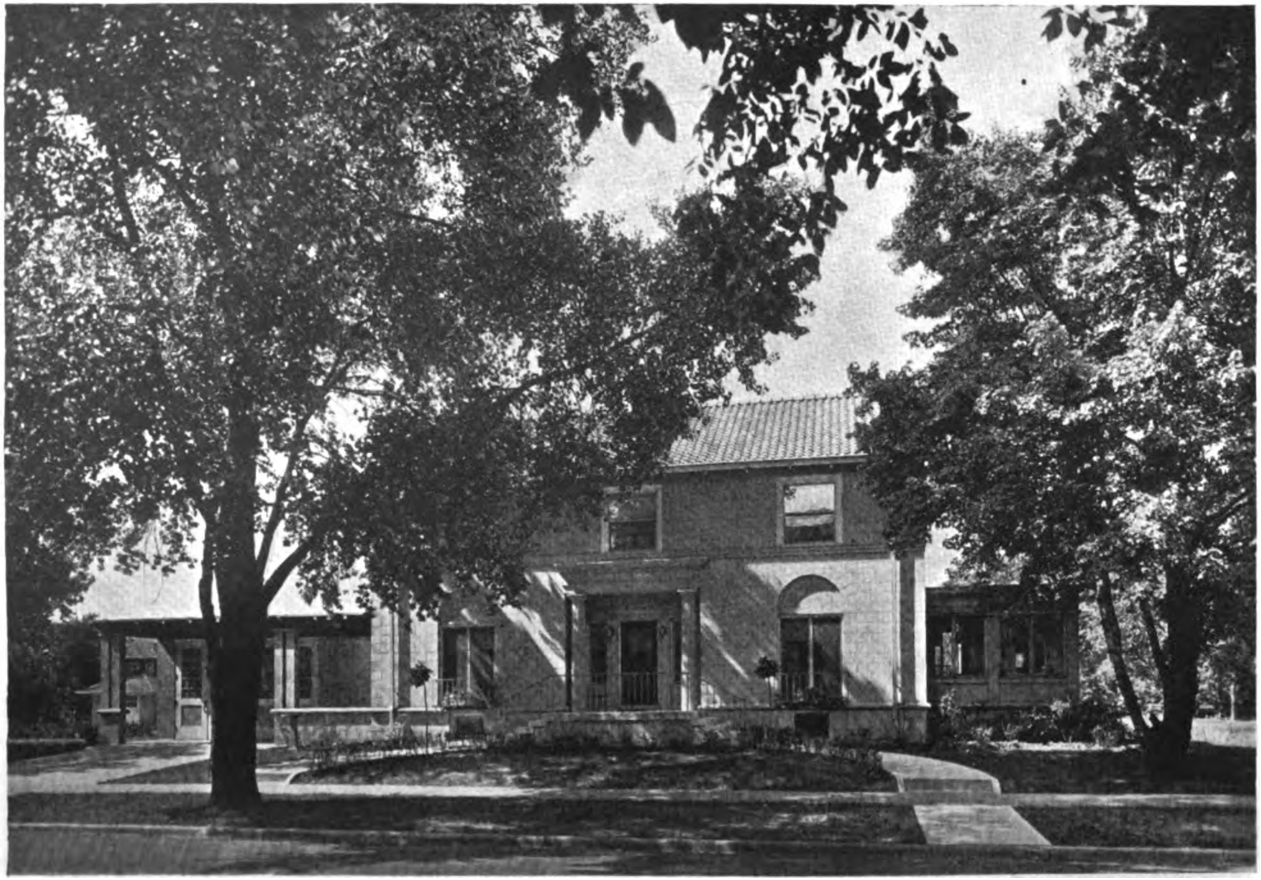

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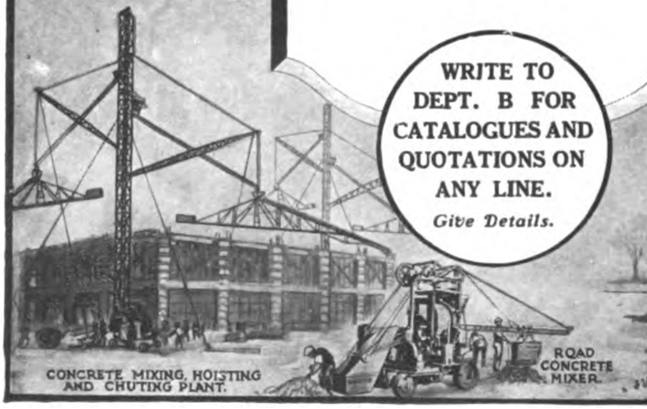
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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XVIII. No. 6. LONDON, JUNE, 1923.

EDITORIAL NOTES.

BEAMS ENCASED IN CONCRETE.

SOME interesting tests carried out to determine the amount by which stresses in steel beams are reduced by the concrete in which they are encased for fireproofing have been carried out by Professor H. M. McKay, Professor P. Gillespie, and Professor C. Leluau on behalf of the Canadian Manufacturers' Association. The tests were carried out on two identical panels carrying 10 ft. by 16 ft. floor slabs 4 in. thick. The slabs and the fire-proofing of the joists were of 1 : 2 : 4 concrete of moderately wet consistency, and placed in accordance with ordinary practice. An account of the tests in *The Canadian Engineer* states that they proved that the flange stress due to the dead and normal live loads together was about 36 per cent. less after concreting than before. The stress due to the dead load and a live load 50 per cent. above the normal, acting together, was some 20 per cent. less after concreting than that due to the dead and normal live load before concreting. Finally, after concreting, the beams sustained the dead load and twice the normal live load with a smaller flange stress than that due to the dead and normal live load before concreting. As present-day practice is to design steel joists encased in concrete supporting concrete floor slabs to take the entire load, including the weight of the concrete, it would seem that considerable economies are possible in this direction.

THE TREATMENT OF CONCRETE SURFACES.

A DISTINCT advance in the surface treatment of concrete is marked by the "Recommended Practice for Treatment of Exterior Surfaces of Industrial Reinforced Concrete Buildings," prepared by a Committee of the American Concrete Institute, an abstract of which is given elsewhere in this issue. In the United States a great deal of attention has been given to the æsthetic aspect of concrete construction, and that the results have been satisfactory is obvious from the fact that the experience gained has been sufficient to warrant the publication of a recommended standard practice—the processes used for obtaining these different textures are now considered to be past the experimental stage, and to have justified their inclusion in the standard practice of building. In this country, however, but little has been done to obtain decorative effects on reinforced concrete, or steel-frame and concrete, buildings by any other means than covering the exteriors with another material of more pleasing colour and appearance than concrete as it is left from the shuttering. The variety of attractive finishes which can be imparted to the concrete itself, at little extra cost, is not generally known, and, of course, cannot be realised until there are in this country some examples of the processes from which the results can be judged by personal inspection: what is undoubtedly the cheapest method of imparting a decorative value to a modern commercial building is in danger of being neglected because pioneers are not forthcoming to adopt it.

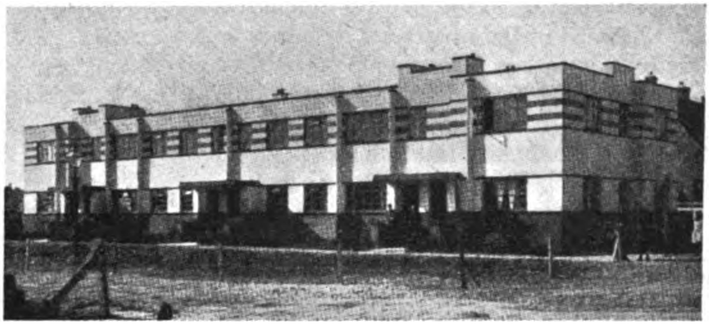
Apart from colour effects, much can be done to enhance the beauty of concrete structures by careful attention to the shuttering in order to get a clean finish, and by fixing strips of wood to the inside of the boards so as to break up large surfaces. An excellent example of this is to be seen at the Wembley stadium, where the entrance to the " Royal Tunnel " has been given the appearance of a stone arch by the use of shuttering which has left indented marks resembling the joints in stonework, with buttresses, voussoir, and keystone. The effect is admirable, and for such a structure could hardly be improved by covering the concrete with any other material, however expensive.

STONE SCREENINGS VERSUS SAND FOR FINE AGGREGATE.

A SERIES of tests to find the relative strengths of concrete made with stone screenings and sand as the fine aggregate were fully described by Professor Duff Abrams at a recent meeting of the National Crushed Stone Association of the United States. In general, the investigations showed that the strength of concrete made from screenings is a little lower than the strength of natural sands. Good concrete, however, can be made with stone screenings as fine aggregate, but it is important that proper inspection be given to see that the material is graded in such a way as to be free from an excess of dust and of extremely coarse particles, and that an excess of mixing water is not used. Due to the angularity of the particles there is a tendency with screenings to use more mixing water than is required by sand, especially where concrete is distributed by chutes. These tests and other investigations show that good concrete can be made from inferior aggregates, if proper proportions and workmanship are employed; but, on the other hand, it is very important to remember that very inferior concrete may result with the best of aggregates if improper methods are used.

BRITISH MONEY FOR BRITISH WORKERS.

WE are glad to see that what has to many seemed a gross misuse of public money is to be brought to an end by the Government. We refer to the announcement of the Ministry of Health to the effect that it is to be a condition of all grants given by the Unemployment Grants Committee in respect of works put in hand expressly because of local unemployment, or by any Government Department, that all contracts for or incidental to the works shall be placed in this country. The Government further urges that, in the absence of special circumstances, the same principle should be adopted in the case of all contracts in respect of works carried out by local authorities, whether or not the expenditure or the works is assisted by grants from the National Exchequer or from the Road Fund. It is difficult to understand the psychology of those who, honestly concerned to relieve the distress resulting from the unfortunate prevalence of unemployment brought about by present conditions, have not hesitated to spend abroad a considerable portion of the inadequate funds at their disposal for relief purposes. The necessity for ameliorative measures during these times of trade depression is not denied,



CONCRETE HOUSES AT ROTTERDAM.

The complete break with tradition which characterises the post-war housing schemes in Holland has aroused much interest and a good deal of criticism in this country. This illustration shows one of the latest blocks of concrete houses erected in Rotterdam, from the design of Mr. Van Hardeveld.

and no one questions the enormous advantages of spending the money available on useful works as compared with the demoralising effect of the "dole." But when we cannot afford to find work for all our unemployed it seems worse than folly to send abroad any of the money so allocated unless it is absolutely necessary. We recently saw the extraordinary spectacle of two towns in the Midlands threatening reprisals on each other because one of them had bought a foreign consignment of a product which could have been obtained, at a figure only about 5 per cent. more, from its neighbour, which was supporting a large body of workers in that particular industry out of the rates—and the town which felt aggrieved at this unneighbourly action angrily intimated that when it required any of the goods the manufacture of which formed the staple industry of the offending town, it also would buy in a foreign market!

Economy is an admirable trait whether in an individual or a public body, but in these difficult times the local authority which, to save comparatively small sums, benefits foreign countries with money which is specially ear-marked for our own unemployed, is laying itself open to being charged with narrow-mindedness and lack of patriotism; it certainly should not have required Government action to prevent money specially allocated for the relief of distress in this country from going abroad.

The announcement of the decision of the Government has not been given the publicity its importance, both to British industry and to the unemployed, deserves, but it should have far-reaching results in checking what appeared to be a growing abuse to which we have frequently called attention.

A VISION OF THE FUTURE.

A STORY which reads like a dream of a harassed concrete designer or constructor was unfolded recently in a paper read at the Rockefeller University, Pekin, in which two Chinese investigators, Mr. Wung chia pin and Dr. Chang yu hsia, related their discovery of a wonderful set of ants, which can produce cement and concrete without any of those aids (and difficulties, we might interpolate), which are thought to be essential for man-made concrete. The author found that these ants, on the banks of the Yangtze-kiang, have formed a concrete honeycomb about 100 yds. wide and up to 150 ft. deep, and of remarkable strength considering how thin are the walls of the numerous chambers and passages.

The ants appear to be of two kinds: (*a*) those chiefly engaged in building and repairing the structure, and (*b*) those concerned with the general welfare of the colony. The former may be divided into four groups: (1) Transporters, which bring the particles of sand, lime, and clay into the structure; (2) the cement makers, which appear to have a very saline saliva, which converts limestone into a material closely resembling eggshells—the exuded material is laid on moist clay ready to be treated by (3), the kneaders, who mix the materials thoroughly together and impregnate them with a further quantity of saliva containing a powerful ferment, which is of such a nature that when the pellets of material have been stored for some time in a warm place they are converted into a cement. The storage and fermentation of the material thus serves the same purpose as heating a mixture of chalk or limestone and clay to a bright red heat, which is the human method of manufacturing cement. How, or why, this saliva should exert a chemical action which can only be obtained by human beings at a red heat is not yet known, but the two investigators claim to have established the fact. The fourth group, or builders, mixes the cement prepared by their companions with sand, and so forms a concrete which appears to be of quite exceptional strength and durability. In order to obtain sufficient quantities of the freshly-made cement, the investigators provided artificial and removable foundations—similar to those used in bee-hives, but much larger—on which the concrete would be erected. When one of these sections has been filled another is substituted.

These ants only work during the season of the plants on which they feed. During the winter they hibernate like bees and other insects; it appears possible, however, to feed them on a preparation of soya bean, and by this means to keep them at work all through the year. It now only remains to train them to take their part in the world's work, and then, until they form their own trade union, those responsible for concrete works will probably be as near Utopia as they will ever be.

Proposed Concrete Bridge over the Seine.

A NEW Pont de la Tournelle is, states *The Times*, to be built to replace the present wooden structure between the Île St. Louis and the left bank of the Seine. It will be the first Paris bridge to be made of reinforced concrete, and is expected to be completed within three years. There will be one long arch of seventy-four metres, and two others of eleven metres each.

WIND: AND ITS EFFECTS UPON STRUCTURES.

By H. F. CORNICK, M.C., A.M.Inst.C.E.

GENERAL REMARKS.

THE importance of wind pressure has been for many years fully realised in this country. Prior to the year 1879 apparently no special provision for the stresses caused by wind pressure was made, and it was perhaps fortunate in one respect that there occurred in that year a disaster which directed attention to this important subject, for with the very rapid increase in the number of iron and steel structures, bridges, buildings, and the like, many more disasters and accidents would very likely have occurred.

The Board of Trade, however, in the year 1881 imposed certain restrictions on the construction of railway bridges, and this was the means of drawing public attention to the necessity for making allowances for wind on structures. The Board of Trade recommendations are to this day in use for bridge calculations for railway work. Later, the London County Council imposed similar restrictions on the construction of iron and other buildings.

There is a tendency nowadays to consider the Board of Trade recommendations too high, and in the light of investigations which have been made during recent years this appears to be fully justified. A pressure of 56 lb. per sq. ft. has, so far as the writer's knowledge goes, only once been officially recorded in thirty-nine years, and this was at the Forth Bridge and was recorded momentarily on a dynamometer gauge. American and German practice is to provide against a maximum pressure of 30 lb. per sq. ft.

The fact that many structures are still standing which were built many years ago without special wind bracings is at first sight somewhat disconcerting. The supposition that the structures have never been subjected to a high wind pressure is probably correct in many cases. On the other hand, it must be remembered that structures such as bridges and the like were in those days constructed of wrought iron and very largely of cast iron; their weight was, therefore, far in excess of similar structures of to-day's manufacture, and consequently, from the overturning point of view at any rate, the inertia of the mass to be moved was considerable. It has been found that high velocities and pressures of wind are not usually of long duration, and therefore would have but little effect upon the mass of the structure.

Again, the rigidity of rivetted connections to a great extent provides strength for resisting wind pressure, which would not have been taken into account in design prior to the later 'seventies. Further, factors of safety were in many cases higher than those in use to-day, and, moreover, the size of bridges and buildings has greatly increased since the advent of steel structures.

These facts may to a large extent explain the seeming anomaly referred to, but at the same time it is needless to remark that nowadays it is not good practice to calculate that the weight only of a steel-frame structure is the sole resisting force opposed to the overturning effect of the wind. In large bridges and buildings now built in steel and reinforced concrete the stresses due to wind pressure are very considerable, and an accurate estimate of the wind pressure and the stresses induced therefrom is essential.

Sir James Wolfe Barry, in his address to the British Association in 1898, said, "It is difficult to imagine the amount of money which has been wasted in unnecessary provision against wind strains of 56 lb. per sq. ft. on large areas in consequence of hurried generalisation from insufficient data."

Sir Benjamin Baker has calculated that in some of the members of the Forth Bridge the maximum forces due to dead load, live load, and wind pressure are as follows:—

Dead Load . .	2,282 tons.
Live Load . .	1,022 ,,
Wind	2,920 ,,

Thus the stresses in this bridge due to wind pressure of 56 lb. per sq. ft. very nearly equal the dead and live load stresses combined, and it must be remembered that the wind is responsible for a considerable portion of the dead-load stresses. Since the erection of the Forth Bridge records have been taken of wind pressure, and the results are very much below the pressure allowed for in its design. It is hoped that very shortly the allowance of 56 lb. per sq. ft. as a maximum for bridges will be reduced to a figure more in accord with recent investigations.

The whole question of wind pressure resolves itself broadly into three heads, as follows:—

(1) To estimate correctly the maximum velocities of wind to be resisted.
(2) To express this velocity as pressure on a body of fixed form, shape, and dimensions.
(3) To calculate the force exerted on structures of various shapes and forms by the wind of maximum intensity expressed as a unit pressure, as in (2).

In these notes the writer has endeavoured to present a comprehensive review of the best experiments and investigations which have been made by authorities on wind pressure in such a manner as to consider the subject from the points of view of the three heads mentioned above.

The writer advocates a reduction in the maximum pressures of wind which are now generally in use in this country, thereby reducing the factor of safety which the present use of high pressures involves—such a factor of safety being at best a clumsy method of covering ignorance—at the same time making the proviso that further research under (3) is necessary before much alteration on these lines can be effected.

OBSERVATIONS OF WIND.

Mr. D. H. Remfry, in his paper on "Wind Pressures and Stresses caused by the Wind on Bridges," read before the Institution of Civil Engineers on February 6, 1923, says; "Taking fourteen important stations in the British Isles for a period of 14 years, from 1890 to 1904, the maximum mean velocity recorded only reached the following values:—

"77 to 80 miles per hour on one occasion
73 to 77 ,, ,, ,, ,, ,,
70 to 73 ,, ,, ,, 5 occasions
65 to 69 ,, ,, ,, ,, ,,

"The records for seven other stations in the British Isles for the 14 years between 1899 and 1913 showed only on two occasions a maximum mean velocity reaching 64 to 75 miles per hour. At the time of the Loch Leven accident, when a train was blown over by the wind, the velocity reached 71 miles per hour, and 73 miles per hour was indicated at Barrow, some little distance away. At the time of the Tay disaster the mean velocity recorded at Glasgow was 52 miles per hour, while at Aberdeen at the same time 70 miles per hour was registered for 5 minutes. It may be assumed that 75 miles per hour is the maximum mean wind velocity in the British Isles which is practically never exceeded."

On the other hand, velocities of 100 miles per hour and over have been officially recorded on other occasions; these velocities, however, would seem to have been over-estimated. They were taken with the Robinson anemometer, the velocity of the cups of which was originally multiplied by 3 to give the actual velocity of the wind. This figure was found in 1902, by further experiments, to be too high, and a figure of 2·2 was substituted. It would appear very possible, if these high velocities were deduced by the coefficient of 3, that the Committee appointed by the Board of Trade to consider the question of wind pressure on railway structures was misinformed as to the possible velocities of wind which might occur. Therefore the maximum pressure of 56 lb. per sq. ft. should probably have been something like 42 lb. per sq. ft., a much more reasonable figure.

The pressure due to wind has been observed in the following ways—

(1) By measuring the velocity of wind by anemometer and deducing the pressure therefrom.
(2) By measuring the pressure on plates exposed to the wind.
(3) By calculating the pressure necessary to overturn railway rolling stock, which have actually been overturned by the wind.

The many instructive records of wind pressure taken at the Forth Bridge were obtained with various sized flat plates;

but these are now known to give too high results on account of a factor of error introduced by the inertia of the pressure plates used.

The figures in Table 1, given by Mr. Adam Hunter, A.M.I.C.E., show the maximum results of some of these experiments at the Forth Bridge.

TABLE 1.

Year.	Date.	Pressure in lb. per sq. ft.				
		Revolving Gauge. 1·5 sq. ft.	Small fixed Gauge. 1·5 sq. ft.	Large fixed Gauge. 300 sq. ft.	In centre of large fixed 1·5 sq. ft.	Right hand top of large Gauge.
1884	Oct. 27	29	23	18	—	—
,,	,, 28	26	29	19	—	—
1885	Mar. 20	30	25	17	—	—
,,	Dec. 4	25	27	19	—	—
1886	Mar. 31	26	31	19	28·5	22·0
1887	Feb. 4	26	41	15	—	—
1888	Jan. 5	27	16	7	—	—
,,	Nov. 17	35	41	27	—	—
1889	,, 2	27	34	12	—	—
1890	Jan. 19	27	28	16	—	—
,,	,, 21	26	28	15	—	—
,,	,, 25	27	24	18	23·5	22·0
Average		27·6	29·8	16·9	—	—

The proximity of the earth influences wind velocities—the velocity, and therefore the pressure, of the wind increasing with the height from the ground. This is probably due, as we should suppose, to the dragging or frictional effect which the ground has upon the wind.

Table 2 gives maximum pressures at the Forth Bridge recorded since its erection. The two readings at 214 ft. were taken at opposite ends of the bridge.

TABLE 2.

Year.	Date.	Pressure in lb. per sq. ft. at various heights.				
		50 ft.	165 ft.	214 ft.	214 ft.	378 ft.
1901	Jan. 26	—	15	25	—	65
,,	Nov. 23	—	50	55	55	60
1902	Dec. 13	—	27·5	31	34	18
1903	Jan. 10	15	20	25	27·5	60
,,	,, 31	—	19·5	29	26	65
,	Mar. 18	20	20	25	29	31
,,	,, 21	10	20	20	22·5	54
1904	,, 26	—	20	32	27	52
,,	Dec. 29	—	22·5	22·5	32·5	—
1905	Jan. 21	—	21	30	23	—
,,	Mar. 18	—	32·5	32·5	42	60
,,	Feb. 28	10	22	20	20	38
1906	Jan. 26	15	—	—	—	59
,,	,, 11	10	20	23·5	25	30
..	Feb. 8	10	15	25	25	55
Average		13·0	23·0	28·0	30·0	50·0

Stevenson's experiments give similar results (Table 3), and appear to show that the pressure varies roughly as the square root of the height above ground :—

TABLE 3.

Ft. above ground.	5	9	15	25	52
Velocity in miles per hour	4	6	6	6·5	7·5
	7	17	18	21	23
	13	23	25	30	32
	19	28	31	35	40
	26	32	34	37	43

Observations of wind velocities, while very helpful, are by no means the only data required for the study of wind pressure. There is, among other questions, the very important matter of the relation between pressure and velocity. This, again, was originally overestimated, and records of pressure given years ago must be examined very carefully before finally accepting them as correct representations of pressures which actually occurred. The size and shape of the objects exposed to wind, their height above ground-level, and their situation generally have a very great bearing upon the actual wind pressure on their surfaces. These relationships are dealt with fully later in this article.

With regard to the third method of measuring wind pressure, i.e., by overturned railway vehicles, railway rolling-stock is not often blown over in England or indeed elsewhere ; neither should we expect this, as the pressure of wind at ground level is, as we have seen, very much less than at higher altitudes. On the other hand, railway vehicles appear to have been blown over in France, on the Continent, and in India and many other places abroad, due no doubt to the more stormy conditions prevailing there.

The information obtained from overturned vehicles is very valuable, for the data obtained is of a definite character, the pressure needed to overturn the vehicle being accurately determinable. From particulars of railway wagons overturned on the Southern Railway of France, tabulated by M. Seyrig, it would appear that the pressure exerted on the sides of the vehicles must have been from 26 to 30 lb. per sq. ft., and in one case 36 lb. per sq. ft. There were in most cases other vehicles in the same train which were not overturned by the wind, and which would have required a higher pressure to do so.

Professor Gaudard has also given some particulars of overturned railway carriages on the section of railway between Narbonne and Perpignan in France, a particularly stormy area. From these observations it appears that pressures of from 30 to 50 lb. per sq. ft. occurred.

Mr. Remfry, in his paper previously referred to, gives a most interesting table of particulars of railway vehicles overturned in India, an abstract of which is given in Table 4.

TABLE 4.

Railway.	Date.	Pressure needed to overturn Vehicle. Lbs Ft^2.	Equivalent pressure on a flat Plate. Lbs. Ft^2.	Equivalent Velocity using Coeff. 0·0031	
				Steady Wind, Miles per Hour.	Maximum Gusts Miles per Hour.
5–6 gauge .	1864	28·0	31·0	100·0	115·5
E.B.S. . .	,,	28·3	31·5	100·6	116·0
E.I.R. . .	1902	29·3	32·7	102·0	118·5
E.I.R. . .	1874	59·0	64·0	143·0	165·0
Metre gauge :					
E.B.S. .	1878	26·0	29·0	97·0	111·5
2–6 gauge	1902	27·0	30·0	98·0	113·5

In considering wind pressure it must be remembered that wind is not always steady, and that the maximum havoc and destruction is occasioned by violent gusts, which may be considerably higher in pressure than the mean pressure but only of short duration. In any case, however, to cause vehicles to overturn or bring about other destruction the gusts must last long enough to overcome the inertia of the wagons or structures.

At the time of the Tay Bridge disaster a gust velocity of 89 miles per hour was recorded at Glasgow, and in the case of the Loch Leven accident it would appear that, as the lightest carriages required a pressure of 32 lb. per sq. ft. to overturn them, the wind must have risen to over 100 miles per hour in velocity.

The records of many important stations in the United Kingdom from 1906 to 1913 show that the maximum gusts recorded were :

99 miles per hour—once.
90 ,, ,, ,, —twice.
85–89 ,, ,, ,, —nine times.

In the table 4 it was assumed that a wagon offered a resistance equivalent to 90 per cent. of that offered by a flat plate having the same side area exposed to the wind. The maximum gust velocity was calculated on the assumption that the maximum gust pressure on an object the size of a wagon was only 75 per cent. of the gust pressure on a small plate 1 ft. sq. due to the wind stream fibres of maximum velocity. The equation used for connecting pressures and velocities was taken as $P = 0{\cdot}0031\ V^2$. From our knowledge of the pressures necessary to overturn empty railway rolling-stock, and our observations of wind velocities, it will be safe to assume that the maximum gust velocity rarely exceeds 100 miles per hour in the British Isles.

RELATION BETWEEN VELOCITY AND PRESSURE.

There have been many experiments on wind pressure, among the names of the experimenters appearing those of Isaac Newton, Didion, De Borda, Rouse, Thibault, Hutton, Smeaton, Dines, Duchemin, and, more recently, Dr. Stanton. Different apparatus was used in many cases by each, and there is great lack of uniformity in results.

The following are a few formulæ which have been proposed from time to time to show the relation between velocity and pressure :—

Rouse put forward the formula—

$P = \frac{V^2}{200}$ where P = pressure in lb. per sq. ft. and V = velocity in miles per hour. This formula is sometimes called "Smeaton's," by whom it was adopted.

Didion's formula in British Units is $P = 0{\cdot}0036\ V^2$ in the same nomenclature as before.

W. H. Dines expressed the results of his experiments in the form $P = V^2 \times {\cdot}003$.

The basis of the last formula is as follows : When a stream of air impinges normally on a flat surface whose area is much greater than that of the stream, the change of momentum per second per ft. of air stream is $Mv = P$ or $\frac{Wv}{g} = P$; but $W = wv$, hence $\frac{wv^2}{g} = P$.

Or, expressed in miles per hour by substituting $v = 1{\cdot}466V$ and putting in the value of w, we have—

$$P = \frac{{\cdot}0807 \times 1{\cdot}466^2 \times V^2}{32{\cdot}2} = {\cdot}0054V^2$$

Where v = velocity of wind in ft. per second.
,, V = velocity of wind in miles per hour.
,, M = mass of air delivered per sq. ft. per second in lb.
,, w = weight of 1 cub. ft. of air, say, ·0807 lb.
,, P = pressure of wind per sq. ft. of surface exposed, in lb.

If, however, the section of the air stream is much greater than the area of the flat surface on which it impinges, the change of direction of the air stream is not complete, and consequently the change of momentum is considerably less than the value just obtained (approximately half).

Smeaton, from experiments by Rouse, obtained the coefficient ·005, but later experimenters have shown that this value is probably too high.

Dr. Stanton's experiments give ·0027 for the maximum pressure.

In all cases of wind pressure on flat surfaces the resultant or net pressure on the surface is composed of a positive pressure on the windward side and a suction or negative pressure on the leeward side. These pressures have been found to vary in the case of round and square plates, and Dr. Stanton gives as an average for the resultant pressure on such plates $P = 0{\cdot}0036\ V^2$.

Table 5 gives the pressure per sq. ft. on a flat surface normal to the direction of the wind for different velocities as calculated by the formula :—

TABLE 5.

Velocity in miles per hour.	Pressure in lb. per sq. ft.	Description of Wind.
10	·36	Fresh Breeze
20	1·4	Stiff Breeze
30	3·24	Strong Wind
40	5·76	High Wind
50	9·00	Storm
60	12·96	Violent Storm
80	23·04	Hurricane
100	36·00	Violent Hurricane

SIZE AND SHAPE OF STRUCTURE AS AFFECTING WIND PRESSURE.

As has been noted, the size, shape, and form of the surface or object exposed to wind have a considerable effect on the pressure on such surface or object.

Dealing first with areas, it has been found that the average pressure on a large surface is not more than two-thirds of the pressure upon a small area when the wind is at all variable; and, as this is a wind which is most usual, this fact is of considerable importance. With a steady wind there is little or no variation in intensity of pressure due to variation in size of the areas exposed. A series of experiments carried out during many varied wind velocities, with a pressure board 20 ft. by 15 ft. and one of 1½ sq. ft. area, were found to give an average pressure on the large board of 66 per cent. less than the pressure on the small one.

From these results and others of a like nature we might infer that the effects of gusts are very local and that the wind moves in local currents in much the same way as is observed in water in motion. The fact that wind pressure per unit area decreases with the extent of the surface was noted in the eighteenth century by De Borda and Hutton.

There is one reason why we should suppose the average intensity of pressure on a large area to be less than that on a small one. Considering the large area, the flow of air is very much upset by the lateral flow of air away from the front of the surface, and the velocity of the wind for a considerable distance in front of the area would be reduced; there would be a wedge of air which would deflect the advancing wind, so that the component of this velocity at right angles to the surface is undoubtedly much reduced.

It has been found that the effective force on an object exposed to wind is the resultant or algebraic sum of a positive pressure on the windward side and a negative pressure on the leeward side. For a flat plate Dr. Stanton found the negative pressure to be about 0·30 of the total resultant pressure (this was on a small rectangular plate). With a narrow rectangular plate this negative pressure amounted to 0·04 of the total. These figures are only applicable to thin flat plates. Duchemin found that while the pressure on the windward side is unaltered, the negative pressure on the rear of a rectangular shape of any proportions is decreased by increasing the thickness of the plate or length of the object in the direction of the stream of wind.

A noteworthy fact is that in the case of rectangular areas or bars the pressure increases with the ratio of length to breadth. For such a ratio of, say, 50 to 1, the pressure is 29 per cent. higher than that on a square plate of the same total area.

Lattice structures present a very interesting problem. The pressure per sq. ft. on a rectangular lattice is found to be considerably higher than that on a solid rectangle of similar overall dimensions. Dr. Stanton has recorded from experiments that the pressure per sq. ft. of surface "seen in elevation" on a lattice girder 29 ft. long by 3 ft. 7½ in. deep is 26 per cent. higher than that on a rectangular plate of the same shape and area, and 46 per cent. greater than the pressure on a square plate of similar area.

M. Gaudard considered that the wind flowing through the orifices of a grating assumes the stream lines of the "contracted vein." He also assumed that the effective pressure on the grating or lattice was proportional to the whole area of the grating, minus the combined areas of the contracted streams. He therefore expressed the pressure "P" on a lattice girder by the formula—

$$P = p(s - k\varphi)$$

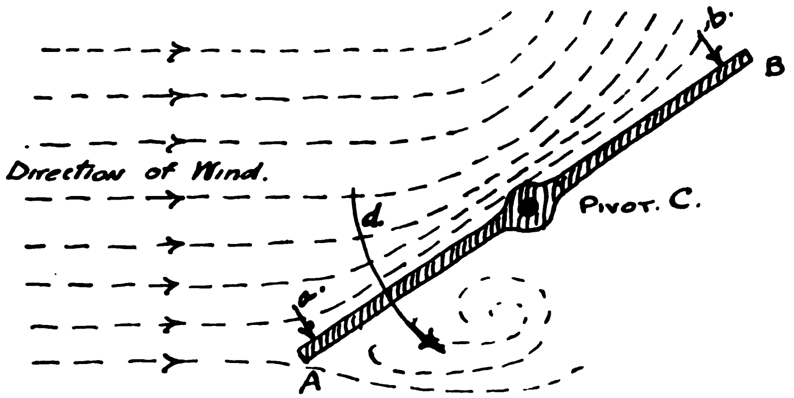
in which p is the wind pressure per sq. ft.
s the total area of the girder or lattice
φ the combined area of the openings
and k the coefficient of contraction.

He also stated that k would be ·65 for small orifices, but would vary inversely as the ratio of the perimeter to surface, and would, therefore, approach unity in the case of large openings.

There is little doubt that wind does assume similar streams as is observed in water passing obstructions, and the form of these "wind streams" has a great effect upon the resultant pressure on various shaped objects and surfaces.

WIND PRESSURE ON INCLINED SURFACES.

It has been found that the distribution of negative pressure on a flat plate exposed to wind normal to its surface was a little greater nearer its edges. In a flat plate inclined at an angle to wind the positive pressure is found to be greater at the forward end than at the rear end. In the sketch, the pressure at "a" normal to the surface of the plane is greater than the pressure at "b" normal to the plane. The plane will therefore rotate in the direction of the arrow "d" around the pivot "c" until it is at right-angles to the direction of the wind. The wind streams would probably assume some such form as is indicated by the dotted lines.



Very similar conditions occur in a roof exposed to wind; the pressure on the roof surface is greater near the eaves than at the ridge; indeed, it seems very probable that there may even be a negative pressure near the latter. It would seem obvious that in the case of roofs the rear side has very little effect upon the effective pressure, except perhaps when the pitch is small.

There have been a great many experiments to determine the effects of wind pressure on inclined surfaces. Hutton's empirical formula, although cumbersome, has been very largely used for the determination of pressure normal to the surfaces of roofs; this formula is as follows:—

$$Pn = P\sin A^{1.842\cos A - 1}$$

where P is the wind pressure on a vertical surface
and A is the angle of inclination of the surface with the horizontal.

A formula by Duchemin is—

$$Pn = P\frac{2\sin A}{1 + \sin^2 A}$$

This gives results somewhat higher than Hutton's.

The Straight Line formula is

$$Pn = \frac{P}{45}A \ (A = \text{or less than } 45 \text{ deg.}).$$

This formula agrees very closely with Hutton's.

Professor Karl Pearson has given a very simple formula. His rule is to take P as 50 lb. per sq. ft. and Pn as as many lbs. per sq. ft. as there are degrees of inclination to the horizontal of the surface, up to 50 degrees, taking the value of 50 lb. for all values beyond. Expressed as a formula, the rule is—

$$Pn = \frac{P \times A}{50} \quad (\text{up to } A = 50, \text{ beyond which } Pn = P.)$$

Dr. Unwin has given the following table of coefficients by which *P* in Hutton's formula can be multiplied to give *Pn*. His table likewise gives coefficients for components, vertical and horizontal to the direction of wind (Table 6).

TABLE 6.

a.	5°	10°	20°	26½°	30°	40°	50°	60°	70°	80°	90°
N	·125	·24	·45	·59	·66	·83	·95	1·00	1·02	1·01	1·00
V	·122	·23	·42	·53	·57	·64	·61	·50	·35	·17	0·00
H	·01	·04	·15	·27	·33	·53	·73	·85	·96	·99	1·00

This Table may be taken as the most serviceable for all kinds of roof design.

In the case of roofs over open sheds in exposed situations there is an uplifting effect which may require attention, for we are all familiar with the upward pull of an umbrella in a wind. The wind should in this case be taken as acting against the inner surface of the building on the leeward side, and the uplifting force can be determined by reference to the table, *Pv* being the coefficient required. With closed gables the effect of the wind is increased from 30 to 70 per cent.

The uplifting effect on close sided buildings is not so very important. An interesting case, however, is that recorded by Mr. Irminger, sometime engineer of the Municipal Gas Works at Copenhagen. He found by means of a pressure-gauge that during a wind which varied from 29 to 70 ft. per second in velocity the pressure inside a gasholder 170 ft. diameter and 90 ft. high fell from 6·25 in. to 5·83. This was equal to a lifting force of 2·2 lb. per sq. ft. on the top of the holder.

WIND PRESSURE ON CHIMNEYS.

We have seen that the effective pressure on a thin flat plate is the resultant of a positive and negative pressure upon the front and rear surfaces respectively, and that the total force varies with the length of the object in the direction of the wind stream. So also does the shape of the object influence the effective pressure. Table 7 gives the relative wind resistances of various shaped objects, as found by experiments.*

TABLE 7.

Flat plate	1
Parachute (concave surface) depth = $\frac{\text{diam.}}{3}$	1·2–2
Sphere	0·36–0·41
Elongated projectile	0·5
Cylinder	0·54–0·57
Wedge (base to wind)	0·8–0·97
,, (edge to wind), vertex angle 90 degrees	0·6–0·7
Cone (base to wind)	0·95
,, (apex to wind), vertex angle 90 degrees	0·69–0·72
,, (apex to wind), vertex angle 60 degrees	0·54
Lattice girders	about 0·8

While the pressures of many of these shapes are of no practical value, they are of theoretical interest. The pressure upon a cylinder with its axis vertical is, however, important from the practical point of view, for this is the form of many structures which have to withstand wind pressure—for example, chimneys, gasometers, circular tanks, silos, coal bunkers, oil reservoirs, piers of bridges, and so on.

It is probable from our knowledge of the effect of wind on an inclined surface, and of the positive and negative pressures involved upon various shapes, that the positive pressure extends over somewhat less than half the circumference; indeed, some experiments have shown it to be less than one-fourth, the rest of the circumference being subjected to a suction or negative pressure. The table of relative wind resistances gives for a cylinder ·54 to ·57 of the pressure on a flat plate. The dimensions of the plate and cylinder in the experiments were more or less the same, whereas the ratio of length to breadth of a chimney is fairly large as a rule and would probably tend to increase the effective pressure, but Rankine gives the following formulæ for chimneys—

---

* See Goodman's *Mechanics Applied to Engineering.*

Square chimney $P = p \times a$

Round ,, $P = p \times \frac{a}{2}$

where P is the total effective pressure against the cylinder,
and p is the maximum intensity of pressure of the wind against a flat surface,
and a is the area of a vertical cross section of the chimney.

German specifications of the year 1908 specify for tall chimneys a pressure of 26 lb. per sq. ft. for rectangular or square chimneys, 67 per cent. of 26 lb. for circular, and 71 per cent. of 26 lb. for octagonal chimneys.

[*To be concluded.*]

NEW DEVELOPMENTS IN CONCRETE SHIPS.

One of the largest concrete ships which has been attempted has just been built at Minden in a large floating concrete dock. It is 223 ft. in length, 27 ft. in width, and draws 8 ft. 6 in. of water when carrying a load of 900 tons, but only 20 in. when empty. A very small draught is always required for concrete ships; one of these vessels built at Minden had a draught of only 8 in. Special efforts have been made to overcome one of the chief drawbacks to concrete ships, viz., their low resistance to shocks and blows, and special fenders of organic material are fitted around those parts of the vessel which are most likely to be struck. The 800-ton barge *Aguilla I*, built by the same firm, is reported by the captain as behaving well. She makes the same movements as an iron vessel. She has had one voyage in exceptionally bad weather, but behaved in a perfectly normal manner.

The floating concrete dock in which both the above vessels were built is also giving entire satisfaction. It is 266 ft. in length, 32 ft. wide, and 12 ft. deep; the walls are 2 in. thick. It required only six weeks in construction. The advantages it has over floating docks made of steel are (i) its low cost (which is less than half that of a steel dock), (ii) greater ease of construction, (iii) much less need for skilled workmen, and (iv) only about half the usual time is needed to build it. Several more concrete docks are also in course of construction, one of the largest being a 30,000-ton dock for the Marine Service.

Exceptionally low costs of construction were incurred in seven concrete floats each from 33 to 50 ft. long and 8 ft. wide, these being used for a bridge for the local authority.

A concrete ferry-boat, 53 ft. by 18 ft. by 3 ft., built two years ago by the same firm, is now in regular use at Petershagen on the Weser. Concrete appears to be particularly suitable for ferry-boats, as it grows stronger instead of weaker with time, and cannot rust or decay. The cost of repairs is much less with concrete than with other vessels, and they can be repaired much more rapidly because they are usually so simple that repairs can be carried out by a member of the crew instead of the vessel being docked.

The extensive use now being made of concrete vessels in Germany shows that this industry is by no means so dead as some technical journals would like their readers to imagine, and as the most serious difficulties now appear to have been overcome there is a definite future for concrete vessels, especially for those working in areas remote from shipyards.—*Tonind. Ztg.*

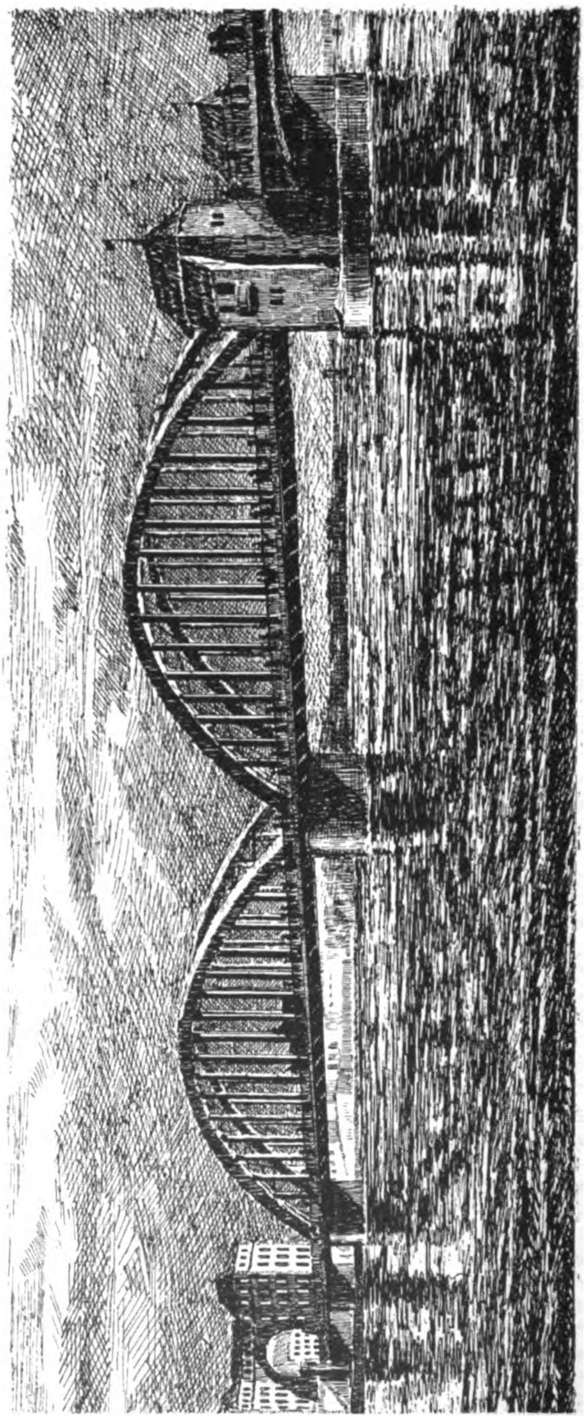


FIG. 7. REINFORCED CONCRETE SCHEMES IN THE INTERNATIONAL COMPETITION OF THE LIMFJORD BRIDGE IN DENMARK. (*See p.* 383.)

REINFORCED CONCRETE SCHEMES IN THE INTERNATIONAL COMPETITION FOR THE LIMFJORD BRIDGE IN DENMARK.

Some time ago the results of a competition for the Limfjord Bridge were published in this Journal without further comment on the schemes submitted ; it will perhaps be remembered that the designs were invited for a permanent fixed bridge over the Fjord in lieu of the existing pontoon bridge which now connects the two towns of Aalborg and Norresundby, situated on the opposite shores of the Fjord. Altogether forty-four schemes were submitted from thirty-eight firms, amongst which three tunnel schemes and seven pontoon bridges were ruled out. The solution of the problem required a high degree of engineering skill in the treatment of the design, on account of the many difficulties facing the designer, both in regard to the foundations, the superstructure and the shore connections, a certain amount of town planning being necessary to bring the approaches and the immediate neighbourhood into harmony with the general appearance of the bridge.

The Fjord is about 1,700 ft. wide at this point, and the nature of the bottom calls for elaborate foundations inasmuch as a 70-ft. deep layer of soft mud has to be penetrated before reasonably solid soil is met ; as the water is about 30 ft. deep the average depth of the foundations is 100 ft. below water level. On account of the difficulty and expense of the foundation work, a long span superstructure was provided for in nearly all the schemes, and, as might be expected in this case, a steel structure was considered the most suitable by the committee of experts formed to examine the schemes. For this reason the three highest prizes were awarded to structural steel designs submitted by German firms, while out of six minor prizes four were awarded to steel schemes and two to reinforced concrete designs.

It is with regard to the latter two that a few explanatory remarks might be of interest to readers of this Journal. That reinforced concrete should appear as a serious competitor in such an important scheme, strongly prejudiced in favour of steel construction by all the conditions at hand, is in itself a pleasant feature and one worthy of note. Altogether five designs in reinforced concrete were submitted, and, as the most important factor in a structure of this description would naturally be lightness of construction, it was to be expected that the designers would endeavour to cut down the scantlings of the main members, the arch rings, by making use of spiral reinforcement and similar means for increasing the compressive resistance of the concrete. The only two schemes that succeeded in reaching reasonable dimensions were, however, based upon the use of Dr. Emperger's patents for cast-iron reinforcement, and were submitted by the Danish firm of A. Engelund, Copenhagen, and the Swedish company, Armerad Betong, Ltd., of Malmoe.

The principle of employing cast iron as compressive reinforcement was described in this Journal some time before the war when Dr. Emperger first published his experiments and results on this new departure in reinforcement. The scheme submitted by A. Engelund was accompanied by the tabulated results of a series of experiments carried out by the Danish State Railways to ascertain the behaviour of the cast-iron reinforcing members in comparison with mild steel reinforcement, in structural members subject to compression, or combined compression and bending, such as would be the case in arch rings. The tests confirmed the claims of Dr. Emperger to the effect that the apportioning or transmission of the compression in a composite material made up of concrete and metal is very much in favour of employing cast iron ; the same considerations that lead to the adoption of the factor 15 generally used for transforming steel areas into concrete would result in a very much higher factor being justified in the case of cast iron ; the average figure arrived at during the tests is given as 42·5. This high value shows how intensively the cast iron is partaking of the compression in a composite member, where, in the nature of things, the various materials composing it must needs undergo the same deformations. Some very important results are

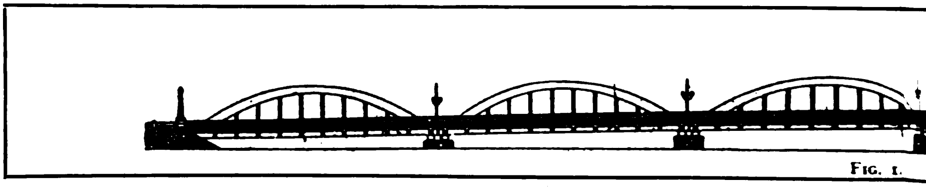


FIG. 1.

deduced from this circumstance; to start with, the amount of concrete in a compression member is reduced very considerably, thereby reducing the weight in the same ratio, so that for this reason the cast-iron concrete approaches the steel structure for lightness in scantlings and ease of handling. A further consequence is the very short time required before the centering can be struck, the cast iron doing such a great proportion of the work of the concrete; this subject has been very carefully investigated by Dr. Emperger, who has published the results of exhaustive trials on the combined strength of cast iron and concrete members before the concrete is old enough to have developed its strength. All these facts lead to some very striking possibilities in respect of centering and strutting, and in both the schemes submitted the whole question of centering was treated with these properties in view.

The general outline of the Engelund scheme is shown in *Fig.* 1, while *Fig.* 2 indicates the proposed method of centering the arch ring and other constructional members. The span between the piers is formed as a parabolic arch with tie, the length of the span being 115 ft. and the rise 21 ft., from which the decking is suspended at tie level as a slab supported by longitudinal and cross beams, while the footpath is cantilevered out on each side of the main deck. The strutting for the falsework is in the shape of similar parabolic arch rings in constructional steel or composite steel and timber, somewhat larger both in span and rise, so as to allow the centering proper for the concrete members to be arranged inside; these temporary arch rings are braced against one another at the top and bottom. The moulds for the concrete arch rings and the bottom ties, slabs and beams, with their suspension members, are supported from the temporary arch rings by means of joists carried by suspension rods from the arch, and are so arranged that the whole of the arch centering can be removed a few days after concreting and erected for use in the next span. The foundation provided for in this scheme is a novel combination of piling and caissons, the latter extending down to mud-level and resting on pile clusters. The piles are cylindrical with an external diameter of 30 in., and are intended to be driven at an incline from a temporary staging, after which the caisson is lowered over the top of the piles and concrete filled in to form a capping and connect the pile heads. *Fig.* 3 shows the caisson with the arrangement

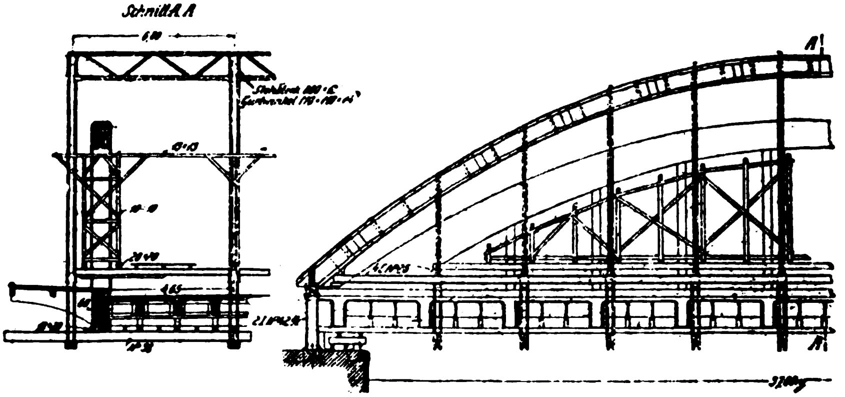
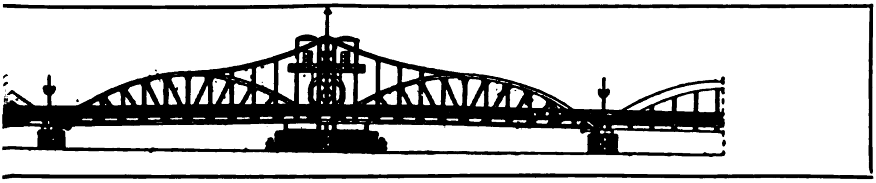


FIG. 2.



for pouring concrete from the top down into the bottom compartment through two vertical shafts.

The scheme by Armerad Betong, Ltd., which is shown in elevation and plan in *Fig.* 4, while some details are given in *Figs.* 5 and 6 (see pp. 416–7), is boldly conceived, well considered and shows the touch of the expert. The arches are introduced in spans of 265 ft. and, as will be seen from the illustration, look slim and well proportioned. By two spans of arches on each side the central opening with a bascule bridge is connected with the approaches, which are formed as a slab and beam design on piers. The arch rings are calculated as two-hinged arches with ties, and the cast-iron reinforcement is shown clearly in the sections. On account of the height and comparatively small section of the rings, the latter are braced together at the top for the greater part of their length. The tie is composed of twelve flats suspended from the arch ring by hangers which also carry the cross beams and cantilevers forming the support for the decking and footpath. In regard to erection this firm proposed two alternatives, the one being the more or less stereotyped application of a temporary pile staging on which the centering is supported, while the other contains some new features. According to the latter method the concrete arch rings, together with the main constructional parts, are to be constructed and erected on shore and afterwards floated out in position by means of two reinforced concrete pontoons, and finally deposited on the piers. During this procedure the structure would be stressed somewhat differently from what would be the case in the ordinary way, and these stresses, of course, have to be provided for in the design. This method of transporting and depositing the bridge members is very similar to that employed in the erection of pontoon bridges. The foundations in this scheme are all of the caisson type for the bridge proper, while the approaches are piled in the usual way, the soft ground extending inwards for a long distance.

Fig. 7 (p. 382) is a view of part of the proposed bridge, prepared with a view of giving an idea of the general appearance of these large arches.

Altogether the scheme submitted by Armerad Betong, Ltd., may be said to have made some bold strides in the design of long span arches, and by duly impressing the committee with the possibilities of concrete construction, it has, together with the other premiated design, done good service in the cause.

R N. S.

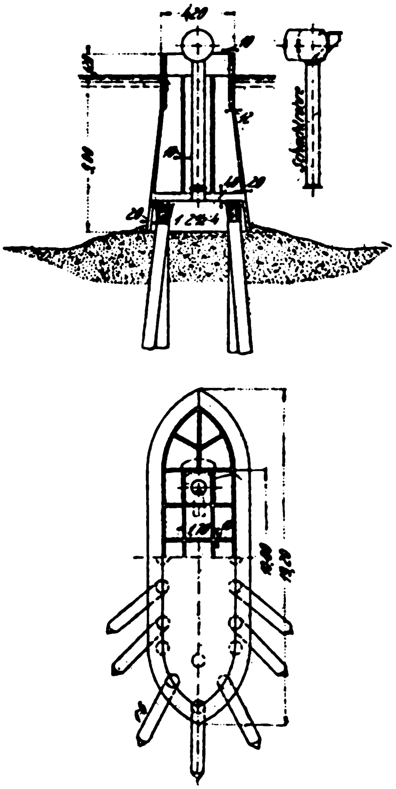
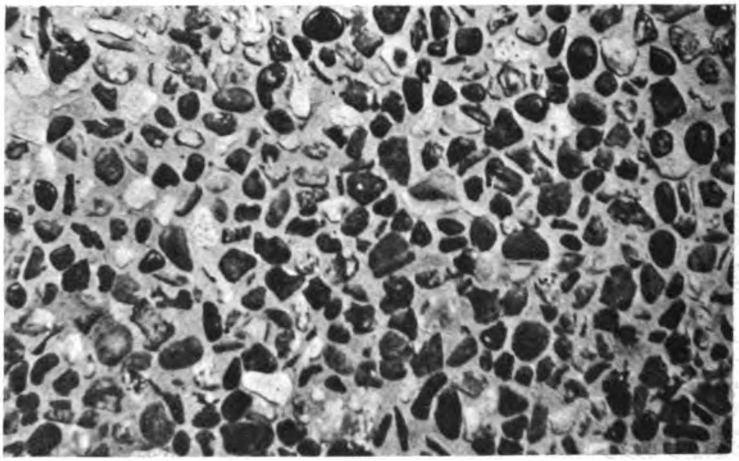
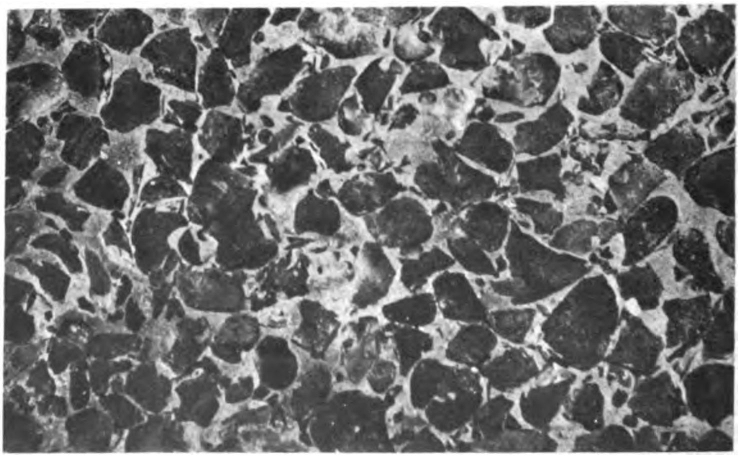


Fig. 3.



CONCRETE SURFACE WITH PIT BALLAST AGGREGATE EXPOSED (One-third actual size).



CONCRETE SURFACE WITH CRUSHED BALLAST AGGREGATE EXPOSED (One-third actual size).

SURFACE TREATMENTS FOR CONCRETE. (*See p.* 387.)

SURFACE TREATMENTS FOR CONCRETE.

COMMITTEE of the American Concrete
te which has considered the ques-
t the treatment of concrete surfaces
issued its report. The different
ents recommended are divided
ve general classes, namely, (1)
g and patching; (2) Correction
mn and beam lines, joints, etc.;
Cement washes and proprietary
s; (4) Rubbed finishes; (5) Tooled
es.

ne section on pointing and patching
lines what might be described as the
nimum cleaning up of the surface
cessary to make a structure at all
esentable, such as filling bolt holes,
bbing down any ridges left where the
huttering has not fitted closely, etc.
he processes mentioned in this section
re, in fact, those which would in any
ase be seen to, apart altogether from
he æsthetic aspect, for without they were
arried out no reinforced concrete building
ould be considered a structurally sound
job.

The second section goes a step further, and deals with matters which again are attended to as a matter of course on practically all concrete buildings, such as protecting any reinforcement which has been allowed to protrude, cutting to line any columns which are out of plumb or beams which sag, and rubbing down small projections. In this connection it is pointed out that it is better, quicker, and cheaper to pound off such projections with a flat-headed hammer than to cut them away with a chisel.

CEMENT WASHES.

The application of cement washes and concrete paints is dealt with at some length, and it is pointed out that cement washes of practically any colour from cream to grey can be prepared by varying the proportions used of light coloured or grey cement and light or dark sand. fter the cement and sand in the desired oportions are thoroughly mixed in the y state, the mixture should be added slowly to water, stirring vigorously until the consistency is that of a stiff oil paint. The dry batch should be large enough for a full day's work, but only enough of the wash should be prepared to last one hour. The wash should be fully stirred in the container before each application to the concrete surface, and when refilling the container all the old wash should be cleaned out and discarded. The area to be coated should first be thoroughly wet, and then a full brush coat of the wash applied. This coat should be rubbed in with a cork float, the surface being sprinkled with a little additional water if necessary. Finally the surface should be gone over with a clean damp brush, brushing in the direction of the board marks. In this process all excess material should be removed, and the remaining coat should be as thin as will permit the surface to be entirely covered. In warm or drying weather the finished surface should be sprinkled with water once a day for three days, and in cool, damp weather should be sprinkled at least once within 24 hours after finishing.

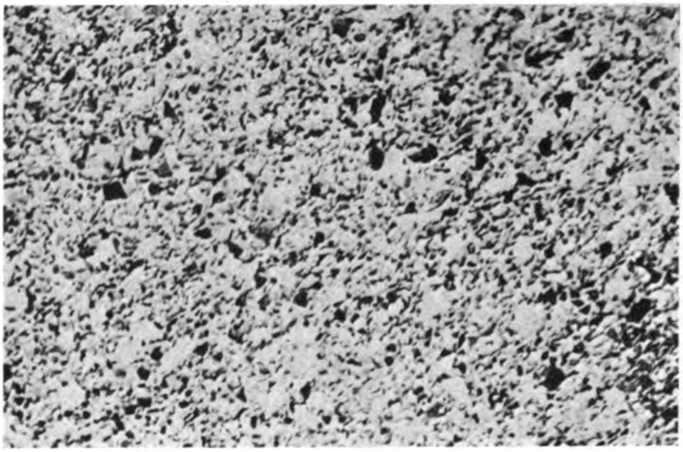
The sand must be dry before screening, not only to facilitate the screening, but also to avoid the possibility of moisture in the sand causing a set in the dry mix. Best results seem to be obtained with a No. 18 sieve.

A very fine appearance is obtained if the wash is rubbed in with a carborundum stone. This not only ensures a better bond, by more positively forcing the material into the pores of the concrete, but at the same time grinds down any slight projection, leaving a semi-rubbed surface.

Sprinkling of the freshly-coated surface is necessary, for if the wash dries before it has attained its set, it will dust off. The sprinkling must be gentle so as not to wash off the fresh coating.

The Committee has been informed that a very effective way of ensuring the bond of the wash coat is to use a 4 per cent. solution of commercial calcium chloride as the gauging liquid instead of plain water. This solution is made up by dissolving 1 lb. of the chloride in 3 galls. of water. The calcium chloride attracts moisture from the air and keeps the wash coat damp for several days, thus ensuring that the cement sets before it dries out.

The cement wash will cost about 1*d.* per sq. ft., and will require about one cubic ft. of cement per 1,000 sq. ft.



Concrete Surface with Exposed Aggregate of Various Coloured Marble Chippings (One-third actual size).

RUBBED FINISHES.

The rubbed finishes are obtained with carborundum stones. It is recommended that the first rub should be completed as soon as the forms can be removed; faces and sides of columns should be stripped in 24 hours if possible, and the soffits of beams should be treated within three or four days. As soon as the forms are removed the surface should be thoroughly wetted and then rubbed with a No. 20 carborundum stone. The rubbing will remove board marks, nail-head marks, and to a certain extent the irregularities between boards.

The cement paste which works up in the rubbing process should be removed by washing and brushing. Small voids in the concrete should be filled with a mortar (usually 1 : 2) composed of finely-screened aggregate of the same general description as that used in the concrete. This mortar should be worked into the face with the carborundum stone and left even and regular.

It is stated that the best and most economical results are obtained by applying the first rub to the concrete while it is green. Filling the small voids with mortar offers no difficulties, but the operator should be warned against leaving an appreciable thickness of mortar on the face of the concrete to take up irregularities in the surface, unless special precautions are taken to ensure the bond. Such precautions are particularly necessary when the surfacing is delayed until the concrete has hardened and dried out.

If the concrete cannot be given the first rub when it is still green, board marks, nailhead marks, and small projections must first be removed. After thorough wetting, the surface should be coated with a cement wash in order to help the grinding action. The carborundum rub should then be given, taking care to remove all the cement wash by washing and brushing.

The second rub should be applied near the end of the work, when the building is ready to clean down and danger of staining from other work is past. The surface should be thoroughly wetted, and then gone over with a No. 24 carborundum stone. The paste which is worked up should be removed with a wet brush, and when dry the finished surface will resemble limestone in colour and texture.

The cost of this process, including miscellaneous pointing and first and second rub, should cost about 3*d.* per sq. ft.

TOOLED FINISHES.

This heading includes all finishes in which the surface of the concrete is mechanically roughened, or removed to expose the aggregate. Wire brushes, stone dressing tools, and rotary cutters are used for this purpose.

When coarse aggregate is to be exposed by wire brushing, the concrete should be as green as possible, usually not over 48 hours old. Generally speaking, this process is not practicable on large buildings unless special effects are to be obtained, in which case the treatment, if it is to be successful, requires careful preparation and selection of aggregates, as well as extreme care in concreting.

When bush hammering or other tooling treatment is to be used, attention should be given to the proper lay-out of construction joints and fill lines, and plans showing the treated areas should be issued before any concrete is placed. The work should be planned to eliminate as many joints as possible, particularly horizontal fill lines. If the latter are unavoidable they should be absolutely level.

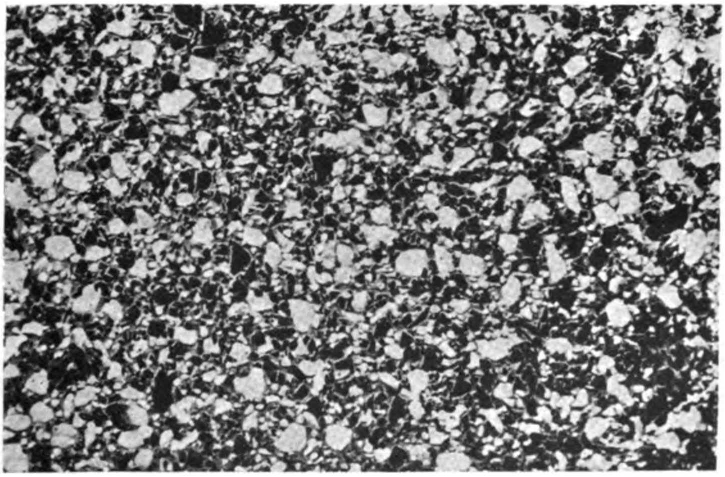
Where coarse aggregate is to be exposed by picking or hammering, it should be of uniform size or uniformly graded. The distribution of the aggregate should also be uniform for satisfactory results, and the consistency of the mix should, therefore, be carefully controlled in order to prevent the settling or segregation of aggregate.

When patching or filling of bolt holes is required on an area which is to receive a tooled finish, the tooling should precede the patching. The latter should then be done by a skilled workman who should match the texture of the surrounding area by imbedding selected pieces of large aggregate in a mortar bed.

Surfaces to be finished with a rough texture should be at least two weeks old before finishing. The work may be executed by mechanical devices operated by air or electricity, or by hand hammering. In general the work on any one panel or area should be done by the same operator in order to avoid differences in texture.

As the tooled finishes are usually carried out in panel effects, great care should be taken to lay out exact lines and to cut clean and sharp at the edges. The borders of the panels and such other parts of the surface as are not roughened must receive a washed or rubbed finish, to balance with the tooled areas.

In the Committee's opinion the effects that are possible under the various tooling treatments are the most pleasing of any. The success of tooling depends upon uniformity in the results obtained, and, therefore, special attention is called to the importance of avoiding joints, fill lines, laitance and segregation. Careful selection of aggregate helps in attaining the desired uniformity, and if the aggregate be selected for colour, its exposure by tooling will introduce a variation of effects in colour as well as in texture.



CONCRETE SURFACE WITH EXPOSED AGGREGATE, BLACK AND WHITE MARBLE CHIPPINGS (One-third actual size).

LIVE LOADS IN OFFICE BUILDINGS.

DURING the latter part of 1922 an investigation was made of live loads on several floors of the Equitable Building in New York City. This structure is well known as perhaps the largest office building in the world, and the varied nature of its occupancy made observations possible which should be very useful in the design of future office buildings and proper regulation of those already in use. The study was instigated by the Department of Commerce Building Code Committee, which is investigating the possibilities of greater economy in structural design, and the objects were to determine the maximum, minimum and average floor loads existing in actual office practice, the nature and distribution of such loads, and the relations which they should bear to the assumptions for design of floor slabs, floor beams and columns as governed by prevailing building practice. With this in mind it was necessary to determine the actual weights of office furniture and merchandise, also their location with reference to partitions and floor beams. In order to cover a wide range of usual office building occupancies, three floors were selected, representing light, medium and heavy classes of occupancy, and the entire area of these three floors was surveyed. Several other offices on various floors, representing what appeared to be unusually heavy occupancy, were also investigated. The whole survey covered over 200 rooms.

The results form a most interesting and valuable collection of data and are believed to be of much significance both to the designers of office buildings and for the purpose of determining proper building code requirements.

The following tabulation shows the outstanding features of the report on this study; Fig. 4 also summarises the observations graphically. It will be noted that little difference exists between the average loading of the floor selected as the heaviest in the building and that which appeared at first glance the lightest:—

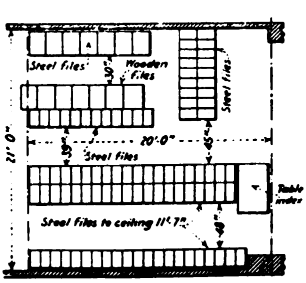


Fig. 1. File Room of Ponding Company

Summary of Loads

	Floor covering, Linoleum	630 lbs
2178	File drawers, 10 lbs each	21,780 lbs
	Racks for same	4,300 lbs
2	Card cabinets, 100 and 300 lbs.	400 lbs
6	Card cabinets, 50 lbs each	300 lbs
1	Flat top desk	200 lbs
7	Steel file cabinets 340 lbs each	2,380 lbs
7	Steel file cabinets, 200 lbs. each	1,400 lbs
5	Wooden file cabinets 250 lbs each	1,250 lbs
5	Step ladders, 50 lbs each	250 lbs
		32,890 lbs Total

Total floor area—420 sq. ft
Load per sq. ft —78 3 lbs

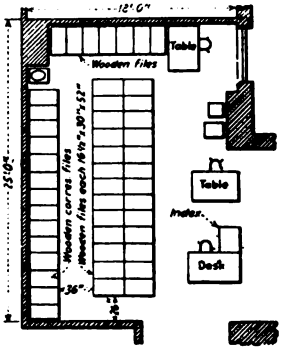


Fig. 2. File Room of Electric Company

Summary of Loads

	Floor covering, Linoleum	675 lbs.
47	File cabinets, 300 lbs. each	14,100 lbs.
2	Tables @ 150 and 100 lbs	250 lbs.
1	Desk	200 lbs.
1	Stand	25 lbs.
1	Water Cooler	90 lbs.
2	Chairs @ 35 lbs. each	70 lbs.
3	Persons @ 150 lbs. each	450 lbs
		15,860 lbs Total

Total floor area—450 sq. ft
Load per sq. ft.—35.2 lbs

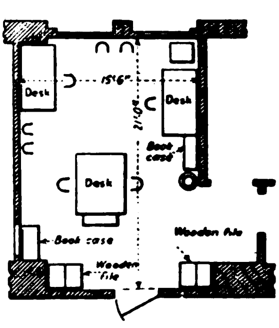


Fig. 3. Office of Electric Company

Summary of Loads

	Floor covering, Linoleum	488 lbs.
3	Flat top desks @ 400 lbs. each	1,200 lbs.
8	Chairs, 3 swivel @ 35 lbs. 5 arm @ 25 lbs.	230 lbs.
2	Bookcases @ 440 lbs. each	880 lbs.
4	File cases @ 273 lbs. each	1,092 lbs.
2	Stands @ 25 lbs. each	50 lbs.
1	Hat tree @ 18 lbs., cuspidors and baskets 30 lbs.	48 lbs.
7	Persons @ 150 lbs each	1,050 lbs.
1	Water Cooler	90 lbs.
		5,128 lbs Total

Total floor area—325 sq. ft
Load per sq. ft.—15.8 lbs

MAXIMUM, MINIMUM AND AVERAGE LIVE LOADS IN EQUITABLE BUILDING.

	No. of Offices.	Maximum lb. sq. ft.	Minimum lb. sq. ft.	Average lb. sq. ft.
Light occupancy floor (20th)	67	55·4	·87	10·26
Medium occupancy floor (37th) . .	64	30·73	3·27	10·67
Heavy occupancy floor (11th) . .	62	33·84	5·00	13·96
Total and average .	193			11·6
Selected heavy occupancies throughout building . . .	14	78·3	21·4	42·4

The weights given do not include the radiators, which averaged 200 lb. each, with two to each bay, located under the windows. These would add approximately 1 lb. per sq. ft. for all exterior bays. The weight of the partitions was not included in the calculations. These in general are 3-in. hollow-tile plastered each side, weighing 30 lb. per square ft., or approximately 350 lb. per ft. run. The weight of occupants was taken at 150 lb. per person.

Examination of bays for which the live load was more than 25 lb. per sq. ft., showed wide variation in the distribution of such loads. The larger proportion was found, as might be expected, within a zone approximately 3 ft. wide around the walls, the remainder being distributed variously in the centres of the rooms. In one or two cases, however, the major proportion of all the load was situated away from the walls, and this condition must be provided for by designers. There is also the probability that practically all furniture may be collected in the central portion of a floor area when occupants are moving, or when painting or cleaning is in progress.

It was found that the heavier loads, such as library shelves and filing cabinets, are likely to be located away from walls and partitions, and the same consideration demands that when the concentrated loads per sq. ft. are excessive, generous unloaded floor areas should be provided adjacent to the concentrations so as to ensure a safe distribution. Among the heaviest occupancies found is the stack room of a large library. The entire floor area is covered with rows of steel stacks placed back to back with 32-in. aisle space between rows, and extending from floor to ceiling, the story height being 11 ft. 7 in. There is a main aisle 4 ft. wide crossing the middle of the room at right angles to the stack rows. The total weight was taken as found to be 45,450 lb. (area, 588 sq. ft.; load per sq. ft., 77·3 lb.) Some of the shelves were partially empty, and the possibility of all shelves being filled should be considered; if this were assumed, the average loading would be increased to 87 lb. per sq. ft.

Only eight articles of furniture (safes) were over 2,000 lb. in weight. A number of sectional filing cases and bookcases with contents weighed much more, but these weights were distributed over such a large area they could not be regarded as concentrated. Of 36 safes and safe cabinets, 23 weighed less than 1,000 lb., 5 between 1,000 and 2,000 lb., 2 weighed 2,200 lb.; 2, 2,360 lb.; 1, 2,800 lb.; 1, 3,000 lb.; 1, 3,500 lb.; and 1, 4,250 lbs. This seems to indicate that if care in regulation of occupancies were exercised considerable economies might be possible in providing for concentrated loads.

The live loads were found lighter next the outside walls. Single-row filing cases, cabinets, safes, bookcases and bins are usually located against blank interior walls. Whether by accident or otherwise, the heavier loads were not found where partitions cut up the floor space into small rooms, indicating that allowance may not be necessary both for removable partitions and heavy floor loads.

Several instances were found where two adjacent floor bays supported average loads of 25 lb. or more, but in no case were two adjacent bays found in excess of an average of 40 lb. per sq. ft.

There are only two or three instances in the floor plans discussed where three offices or store rooms meet at the same column, and it is probable that this condition will be found but rarely in buildings designed for a sufficiency of light and ventilation. In view of this and of the surprisingly light average floor loads discovered, it becomes an interesting question whether in a building of this type further reductions in loads assumed for column design could be made with safety. The building was designed fcr a

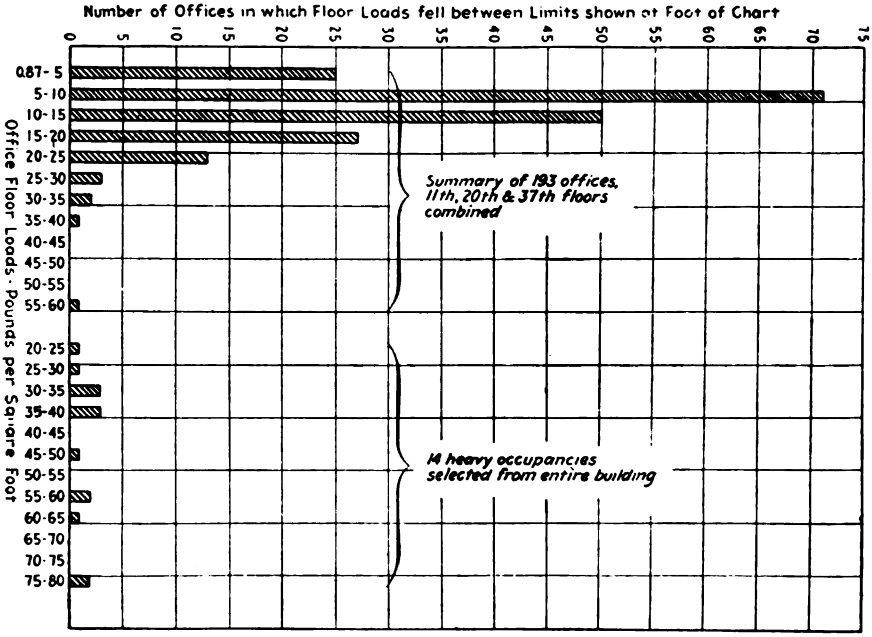


FIG. 4.

live load of 100 lb. per sq. ft. on the first floor and 75 lb. on all others.

A less exhaustive but nevertheless interesting investigation of office floor loading was recently made for the Committee by Captain M. W. McIntyre, manager of the Union Central Life Insurance Company's Building in Cincinnati. This study included several large sections of different stories devoted to clerical and filing purposes. Each individual piece of furniture was not weighed, but the average weight of representative articles was carefully ascertained, and this result applied to furnishings of four floors. All files, desks, etc., were considered as being 100 per cent. full or furnished with all necessary accessories.

The weight of employees, computed at the rate of 150 lb. each, added from ·9 to 1·75 lb. per sq. ft. of floor area.

The tabulated results of these investigations are given in the next column.

Seven individual offices similarly surveyed showed an average weight of furniture of 7·66 lb. per sq. ft. This building, incidentally, was designed to carry a live load as follows: Basement and first floor, 100 lb. per sq. ft.; stairways and corridors, 75 lb. per sq. ft.; other floors, 50 lb. per sq. ft.

	No. sq. ft.	No. pieces of Furniture.	Total Weight of Furniture.	Weight of Furniture per sq. ft.
			lb.	lb.
Section A . . .	10339	635	104478	10·05
Section B . . .	9303	637	27085	2·91
Section C . . .	7348	273	36306	4·92
Section D . . .	10339	702	121388	11·74
Average . . .	9332	561·5	72314	7·405

The Committee has investigated the live loading in a reference room of a New York insurance company equipped with heavy steel filing-cases for large insurance map books. This was considered to represent unusually heavy occupancy. The cases were placed back to back and occupied a floor space approximately 5 ft. by 22 ft. and were 55 in. high. It was found that the case with its aisles for access occupied a space of 247 sq. ft. and weighed, with contents, 10,620 lb. Observation showed it liable to use by no more than 12 men simultaneously. The total load occasioned amounted to 50·3 lb. per sq. ft.—*American Architect.*

CORROSION OF IRON IN CONCRETE.

In a report on the prevention of corrosion of iron in concrete, Mr. Bruno Zschokke, of the Zurich Institute for Testing Materials, recommends the following precautions :—

(1) The concrete should not be too meagre, but should contain the best percentage of cement so as to make it impermeable to air. Thus, the proportion of calcium hydrate, which prevents oxidation, is increased, and the concrete is also more impermeable to carbonic acid, which neutralises the lime.

(2) There must be no substances in the concrete such as locomotive cinders, often containing sulphur, which exert a chemical action on iron.

(3) The layer of concrete covering the iron should be of sufficient thickness properly to cover it, and should not crack under pressure or through shrinkage.

The report states :—

It has been demonstrated by the experiments of Gary that a compact cement or concrete in sufficient thickness will protect iron against oxidation for a considerable length of time, provided that the cracks due to shrinkage do not reach the iron.

The experiments also demonstrated that a coating of red lead on the iron will prevent rust during five years. It must, however, be noted that the concrete employed in the experiments was not of the composition usually used in building construction, but was a mixture of 1 part of cement to 4 parts of sand, perfectly free from cracks.

The problem of the prevention of the formation of cracks in shrinkage has not yet been solved, and as they make their appearance in rich concrete oftener than in meagre, the question arises as to whether oxidation cannot be more surely prevented by other methods than those specified.

It has yet to be ascertained whether the action of solutions of chromates cannot be profitably applied to protect iron in concrete. To elucidate this the point to be examined is whether the effect of these solutions when mixed with certain other saline solutions, or even solid substances, would not be more or less affected or even nullified. This is the case with alkaline sulphates, chlorides, and alkaline earth, e.g., sea salt, calcium chloride, magnesium chloride, Glauber salts, etc. Thus, when iron is exposed to the action of sea-water or salt marshes the use of chromates is out of the question.

The principal systems hitherto adopted to prevent rust consist in the production of a magnetic oxide film in coating the iron with another non-oxidable metal by galvanisation, electrolysis, spraying by the Schoop method, etc., ordinary varnishing, or coating with an oily substance.

The Zschokke method, however, consists in the utilisation of the known property of some salts which makes iron passive when immersed in aqueous solutions concentrated to a degree exceeding a certain limit, usually very low. The most energetic of these salts comprise chromium salts, especially sodium bichromate and mono-chromate.

To ascertain the influence of cement on the protective chromates, especially in mixing, when the calcium hydrate is liberated, a small polished iron plate was dipped into a 0·5 per cent. solution of potassium bichromate with 6 grammes of Portland cement underneath. After a year the plate was covered with a pale grey film of calcium hydrate, without a trace of rust, a proof that the calcium hydrate disengaged by the cement had no action on the chromates. Moreover, the hydrate and potassium or sodium bichromate form a potassium or sodium mono-chromate and calcium chromate, the last mentioned possessing passive properties in a high degree, as demonstrated by the experiments.

Then cements were made with solutions of chromates to test the protective action, and also with ordinary water, for comparison. A polished steel plate was coated with this cement, the weight of the coat being estimated dry and made equal for all the plates of the same series. The other side of the plate and edges were washed over with an asphalte varnish. It was noted, and this is a very important point, that adhesion of the cement to the iron varies considerably with the amount of water. When the cement is too dry it separates from the metal in drying, whilst with a suitable quantity of water (40 per cent. to 50 per cent.) a crust forms on the metal. When frequently covered with water during the first five or six days this crust adheres so firmly that it cannot be detached even by striking it repeatedly with a hammer.

When the cement had hardened the plates were laid with the cement side uppermost in glass dishes containing a layer of water. Then the plates were sprayed once daily with a mixture of equal parts of distilled water and Seltzer water, i.e., water containing carbonic acid. The dishes were then closed with a glass lid, and kept at an ordinary temperature. After a few weeks or some months the plates with ordinary cement were spotted with rust, whereas the others were absolutely free from rust.

According to these experiments iron in concrete can be protected against oxidation in two ways. In the first place it can be coated over with a chromate cement and, when this has set, embedded in concrete in the ordinary way, or, second, the concrete can be mixed with the chromate water.

The first method has the advantage of thoroughly protecting the iron, more so as the thin coat of cement cannot crack when properly applied, and the consumption of chromate is relatively small. Plates thus treated and kept for a year on the roof of the Zurich Institute for Testing Building Materials, exposed to all kinds of weather, were perfectly free from cracks and rust. There is no risk, in this method, of the water in the concrete affecting the chromate in the cement, as demonstrated by the following test. Iron bars, 150 millimetres long and 18 in diameter, were coated with Portland cement and a concentrated solution of calcium chromate, and other bars with the same cement mixed with a 5 per cent. potassium bichromate solution, then left for twenty-four hours to set. They were then put into an iron mould, which was afterwards filled with concrete made with 1 part Portland and 3 parts of normal sand, mixed with 10 per cent. of water. After eight days the two blocks were worked with a hammer and chisel until the thickness of the concrete on the bars was only two to three millimetres. Finally the remaining concrete was filed off from the coating of cement and the dust removed by lengthy immersion in hot water. The filtered water had not the least tint of yellow, a proof that the chromate cement was not affected by the wet concrete.

In the second test the concrete mixed with the chromate solution was used. As yet the results of experiments on a large scale are not available, but it may be taken for granted that the effect is the same and perhaps better than in the first method, as the quantity of chromate is relatively much greater compared with the mass of iron, an important point, already noted by Heyn.

A test was made which demonstrated that chromates have no effect on the setting and resistance of cement.

Only practical experiments, on a large scale, can show which of these two methods is preferable. Consumption of chromate, the price of which is not negligible, is much less in the first than in the second method; moreover, the cement coat protects the iron against rust much better than a meagre porous concrete. On the other hand, coating iron with chromate cement involves expense, whilst the second method requires a greater percentage of protective materials but has the advantage of greater simplicity as the chromate solution supplied by the maker in the proper percentage can be added to the water when making the concrete. The extra expense estimated on the basis of Zurich prices would be 3 to 4 per cent. As previously observed, iron is thus only protected against ordinary atmospheric effects. Acid emanations, for example, would decompose chromates.

CONCRETING A CAVED-IN MINE SHAFT.

In a paper read before the Engineers' Society of Western Pennsylvania on "The Use of Cement and Concrete in the Underground Workings of the North Butte Mining Co.," Mr. Robert Linton states that the use of cement in connection with underground operations has increased rapidly within the past few years. The first shaft in the United States to be lined with concrete was in West Virginia, in 1903. Since then numerous shafts have been concreted, and much concrete used in tunnels and drifts, underground skip-pockets, and other workings where permanence is the controlling factor in the selection of construction design. Cement has also found a wide use underground for fireproofing purposes. The principal mine of the North Butte Mining Co. is operated through three shafts. The main hoisting shaft is the Granite Mountain shaft, 3,740 ft. deep. There are approximately 36 miles of drifts and cross-cuts in the mine, and over 15 miles of track in service underground. In 1916, the general manager of the company used concrete in the "Speculator" shaft, 2,000 ft. in depth. The first work was the construction at suitable points of concrete bearers which form permanent supports for the timbering and prevent any further vertical slipping of the timbers resting upon them. They were built about 6 ft. in vertical thickness, the inside coinciding with the inside of the shaft timbers and extending from there outwards to solid rock (*Fig.* 1). The concrete consisted of 1 part cement, 2 parts sand, and 4 parts gravel, and was reinforced with discarded hoisting cables (1·25 in. diameter) and with old steel rails. The use of these bearers proved

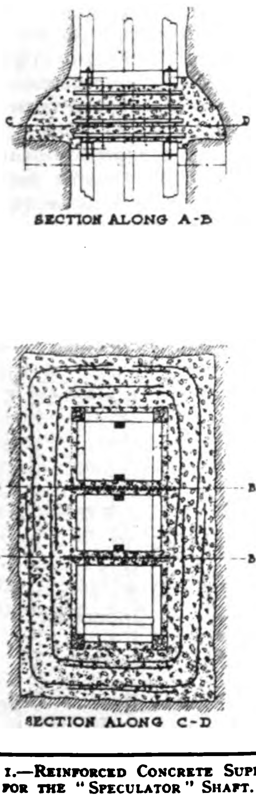


Fig. 1.—Reinforced Concrete Support for the "Speculator" Shaft.

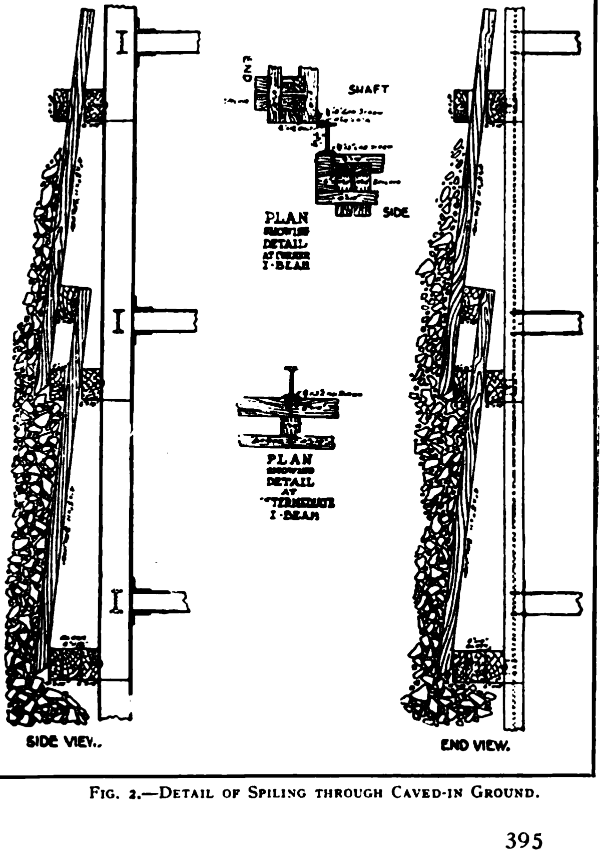

END
SHAFT
PLAN SHOWING DETAIL AT CORNER I-BEAM
SIDE
PLAN SHOWING DETAIL AT INTERMEDIATE I-BEAM
SIDE VIEW.
END VIEW.

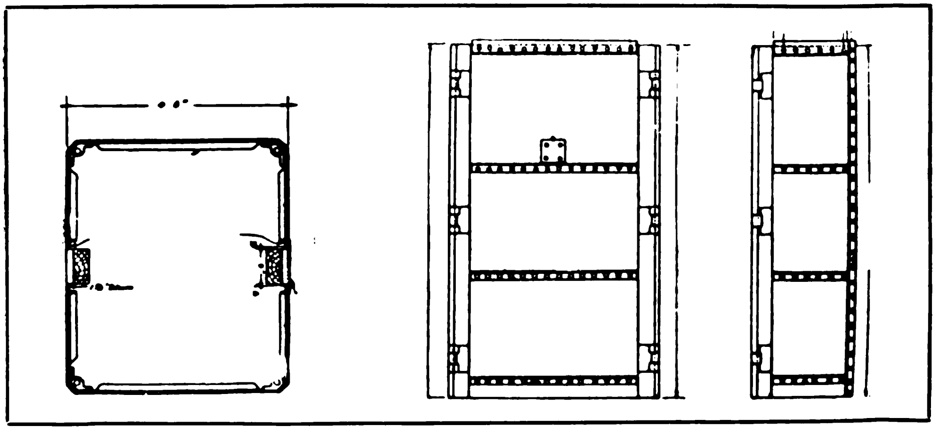
Fig. 2.—Detail of Spiling through Caved-in Ground.

so satisfactory that it was decided to extend the concreting through all the heavy ground. Bearers were placed at convenient points and between them walls were built up 15 in. thick and backfilled with crushed rock This work totalled about 500 lin. ft. The resultant economy is apparent by comparing the cost in 1916 with that of the last six months of 1920. In 1916 there were employed on " Speculator " shaft repairs an average of about 32 men with a total cost for the year of about £10,000. Recalculating this on the basis of wages in effect in 1920, the cost would be increased to an average of £1,150 per month. During the six months ended December 31, 1920, the average force was 2·4 shifts per day, and the total monthly cost of repairs to the " Speculator " shaft averaged only £78.

About the same time studies were initiated to investigate the use of cement in the lateral workings, with a view to determining what saving in cost of timber maintenance could be effected, especially in timbered drifts. These investigations indicated that coating new timbers with cement materially lengthens their life, especially in hot and humid workings. In very heavy ground, however, the timbers are eventually crushed or split and must be replaced, so that practically no economy results from coating them in such cases. Coating old timbers with cement arrests decay when not too advanced, and undoubtedly prolongs the life of timbers so coated. Cement coating of ground forms an effective air seal and, if the cement is applied immediately after the ground is broken and the rock face exposed, there is practically no danger of rock falls occurring. The cement gun is used for coating the timbers and exposed wooden surfaces. Cement does not adhere well to wood in most cases; consequently, before the timbers are coated, expanded metal or woven wire (not coarser than 1-in. mesh) is tacked to them. This acts as a light reinforcement throughout the entire film of cement, which averages about ⅜ in. thick, and ensures its permanent adhesion to the structure.

In 1917, a fire in the " Granite Mountain " shaft destroyed the timbering from the 1,700 ft. to the 2,800 ft. level. The shaft was badly caved for considerable portions of the distance. It was therefore decided to make the repairs by using monolithic concrete for the entire length of shaft in which the timbers had been destroyed. The concreting was eventually extended to below the 3,000 ft. level. Owing to the caved condition of much of the shaft it was essential to design a structure which would possess sufficient strength and rigidity to resist any lateral pressure to which it might be subjected. These requirements were met by building reinforced concrete around a structural steel framework; 6 in. 12½ lb. beams were used for the horizontal members, and 10-in. 25-lb. beams for the vertical members, connections being ⅜-in. angles and splice-bars riveted to the beams with ⅝-in. rivets.

The steel structure is carried on solid reinforced concrete bearers placed at points in the shaft where the ground is



Figs. 3 and 4.—Details of Steel Forms (Plan and Sections).

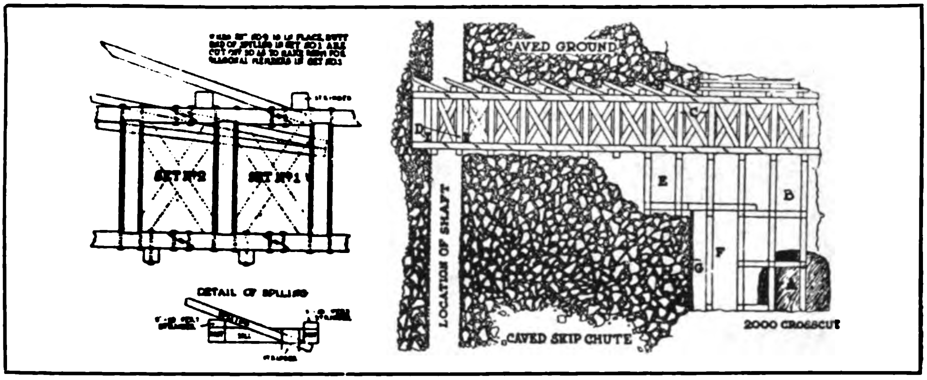


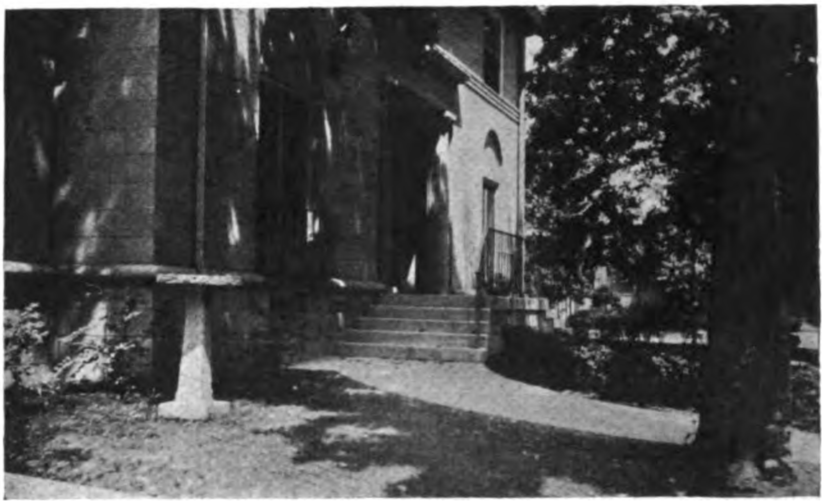
FIG. 5.—DETAIL OF TRUSS.

firm. Between bearers the concrete shaft lining varies in thickness from 15 to 20 in., according to the character of the ground. The four compartments of the shaft are separated from one another by solid partitions. For a few short stretches these partitions were extended to form ribs bracing the structures against the solid rock. The caved ground around the shaft proper was secured by driving overhead and side spiling, and the shaft cleared of débris on the station level. Sinking through the caved rock and débris was then begun, and structural steel sets placed as fast as there was room to hang them. Much of the ground required spiling, which was driven from 6-in. by 10-in. timbers resting against the steel (*Fig.* 2). In the heavier ground the steel sets were braced with timbers which were removed as the concrete was poured. The concrete was mixed on the level above and handled through 6-in. pipes to the point of use. The maximum distance the mixed concrete was spouted was 400 ft.

The most difficult situation was on the 2,000 level. Here the station was entirely caved to a height of over 50 ft. above the sill, and the cross-cuts leading from it were caved for a total combined distance of 150 ft. Furthermore, the shaft walls were caved for some distance back. To reach the shaft, a timbered raise was run through the broken rock to the solid rock 50 ft. above the sill, and a passageway was timbered to the shaft. To support this heavy ground, bridge trusses of timber were built (*Fig.* 5). By slow and careful work the ground was finally spiled, and the steel placed to a point 66 ft. below the level where the rock was sufficiently firm to hold a bearer. The shaft here was concreted as in other cases, except that the walls were made 20-in. thick and very heavily reinforced.

The shaft was concreted for a total distance of 1,340 ft. The structural steel was carried down to 100 ft. below the 2,400 level. Below that point the shaft was not so badly caved, although filled with rock and débris, and reinforced concrete alone was used. Of the total of 1,340 ft., a distance of 813 ft. was repaired by using concrete with steel framework, and 527 ft. without the structural steel. There was placed altogether a total of 6,380 cu. yds. of concrete, and, on an average, 118 lb. of reinforcing rods per cu. yd. of concrete set. The entire job was done in twenty-two weeks. Between the top of the monolithic construction and the collar of the shaft, the compartment containing the manway with sub-compartment carrying the pipe lines and electric cables was fireproofed with gunite.

The concrete has now been in place for nearly three years. During that time no repairs whatever have been required, the concrete has not developed any cracks, and is in as good condition as when placed. There seems to be no doubt that it will so remain as long as the shaft remains in service. The length of life of a mine necessary to justify concreting varies with the conditions, but it is believed that with few exceptions there would be a saving if the shaft is to be used for a period of ten years. The economies do not take into account the advantages resulting from having the shaft absolutely fireproof and entirely obviating the risk of fire.



Mr. Henry K. Holsman, Architect.

CONCRETE HOUSE AT CHICAGO. (*See p.* 399.)

CONCRETE HOUSE AT CHICAGO.

THE concrete house illustrated herewith (and also in our frontispiece) has been recently erected at Oak Park, a suburb of Chicago. The building was designed by Mr. Henry K. Holsman, President of the Illinois Chapter of the American Institute of Architects; it contains eight rooms, and concrete units were exclusively used for the walls.

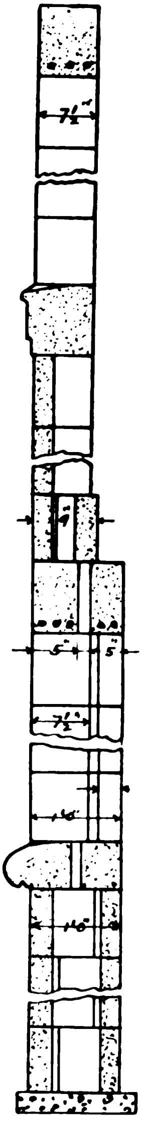
The construction methods followed are those now almost standard for concrete block-work in America. A footing of monolithic concrete 24 in. wide and 15 in. deep was placed below frost penetration, and the double wall of T shape blocks was continued up to the roof lines. The standard blocks have a face 24 in. long and 9 in. high in the wall. The sill and watertable courses are only 6 in. high, but all others are 9 in. high.

All the concrete units used on the exterior of the wall above ground-level were surfaced with a mixture of light coloured Portland cement, white marble, mica-spar, and a sprinkling of black marble, to which was added buff iron oxide to the extent of about 4 per cent. of the volume of the cement, which gives the entire surface a pleasing buff colour. The blocks have flat faces with a rather smooth texture. The mortar used was made of 1 part Portland cement to 2½ parts fine sand, light-coloured Portland cement being used in the mortar for the portions of the work visible on completion.

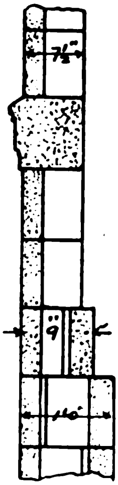
Two large Doric columns at the entrance support some 6½ tons of lintels, etc., above. The watertable and string course at the second-floor level add much to the richness of the exterior lines. The large cast semi-discs over the main window openings on the front elevation are also attractive. These and the panels above the entrance door are panelled to receive appropriate sgraffito designs; these pieces are amply supported by large ears protruding back into the surrounding masonry.

Square columns were used in the sun-room detail, as well as for the supports used at the opposite end of the structure to carry the roof over the passage to the garage. The square panelled chimneys are constructed of L-shape sections designed so that the vertical joints alternate with each course. The main chimney is 16 in. sq., lined with clay

DURING CONSTRUCTION.



SECTION I-I



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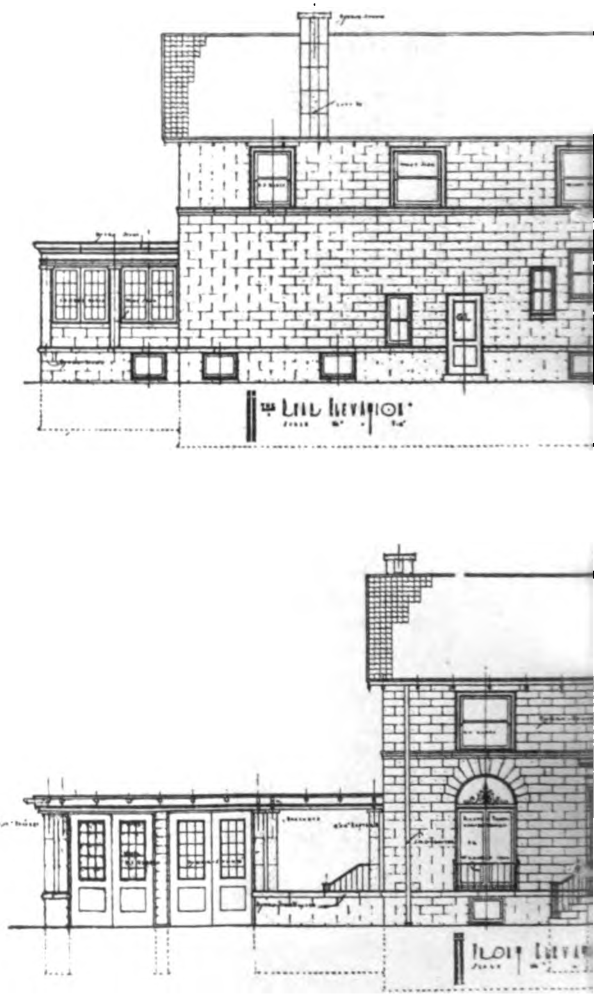
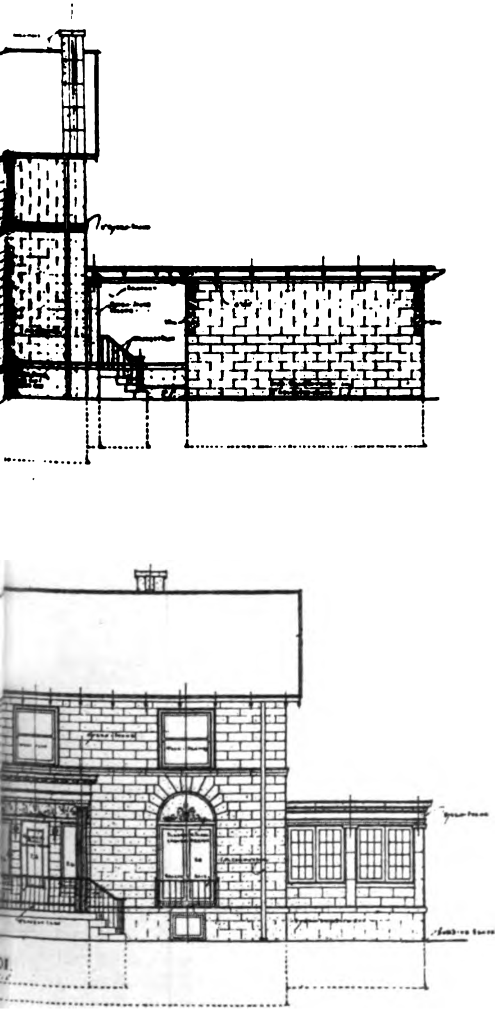


CONCRETE HOUSE A

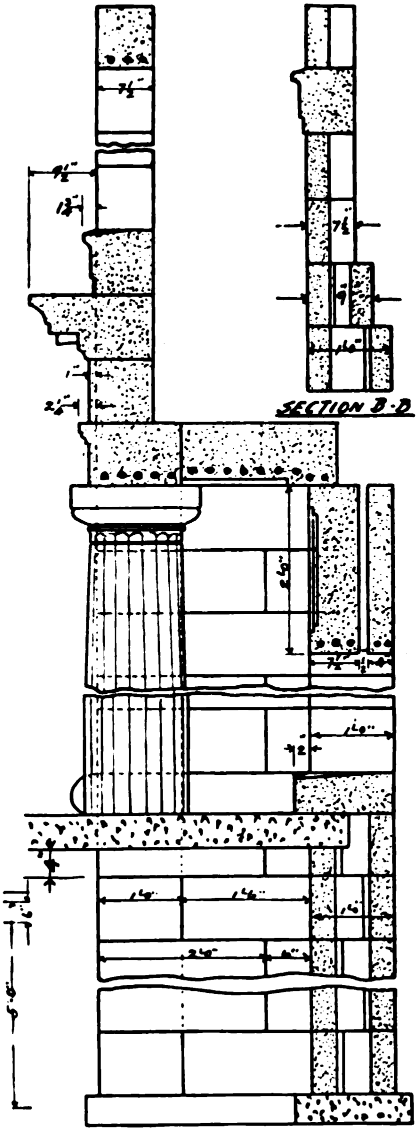
Mr. H. K. Holsman,

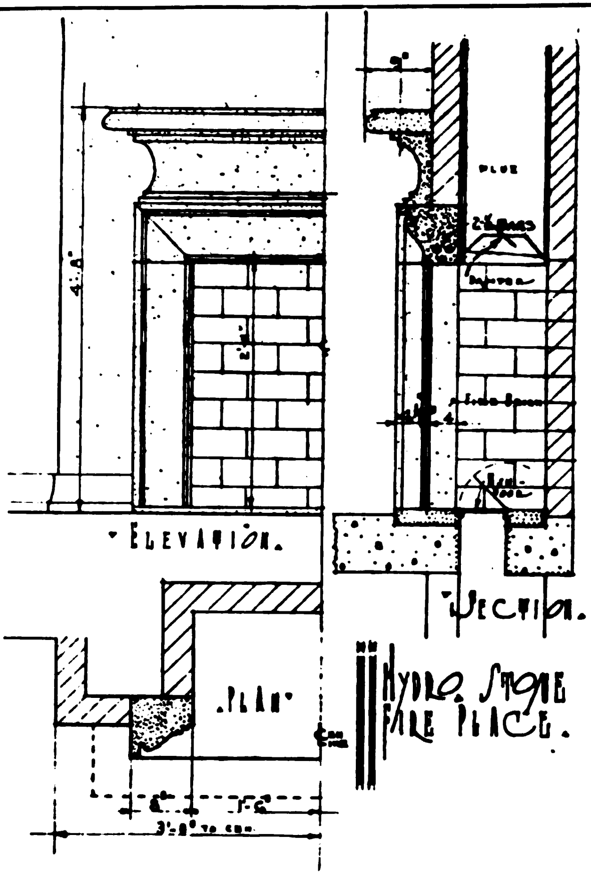
(See p. [illegible])



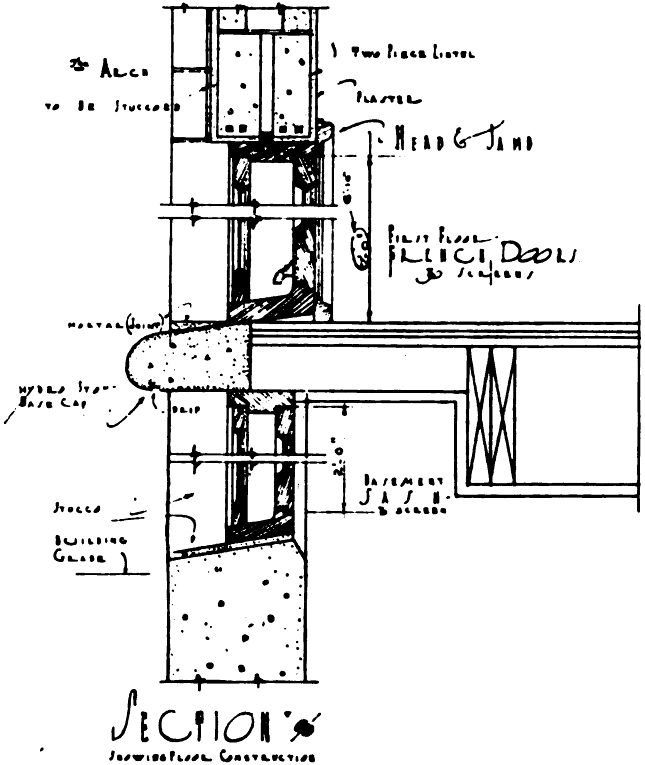
' CHICAGO.

Architect.





Details of Concrete Fireplace.



Section Showing Floor Construction.

Mr. K. Holsman, Architect.

CONCRETE HOUSE AT CHICAGO

e-lining, and provides draught for a large eam boiler, furnishing a vacuum vapour ating system. The other chimney, 13 . square, provides draught for the living-om fireplace. Over 6,600 pre-cast units ere used in the entire structure.

Main floor support is furnished by ncrete-block partition walls, 8 in. thick, the basement, and by the main bearing alls. Wood floors of usual joist con-truction were used throughout, except the sun-room floor and the area bove the heating plant, where rein-rced concrete slabs were used. The n-room floor has a terrazzo, or ground nd polished, surface.

The porch slabs and steps were care-lly laid out and cast in place. These ere all finished with the same mixture light-coloured cement, marble, and on oxide used in the surface of the locks and given a gritty finish to prevent ipping and the formation of hair cracks. In order to add a few delicate lines to the levations, wrought-iron stair rails, window baskets, and lanterns were used.

The roof construction is somewhat unusual in detail. Seven-ply asphaltic roofing paper and zinc flashing were used over a heavy wooden frame, on which were laid Spanish-type dull blood-red clay roofing tiles. Contrary to usual American practice, the gutters are of wood lined with seven-ply paper. Copper down-pipes were used.

A minimum of wood was used in the interior, the principal rooms on the first floor being connected by arched openings without doors or frames. All of the windows are framed in a narrow picture-moulding effect—without aprons or stools.

Sand-float finished plaster was used in all parts of the house, except the kitchen, bedrooms, back hall and bath rooms, and was painted and tinted. Kitchen and bath room walls, above the white tile wainscoting, are covered with oil cloth.

This residence has proved highly satisfactory after an occupancy of several months.

BOOK REVIEW.

Resistance of the Zone of Compression of Reinforced Concrete.* By O. Graf.
Berlin: Wilhelm Ernst und Sohn. Price 1s.

THIS pamphlet of 41 pages, with 43 diagrams, is one of the volumes issued by the German Reinforced Concrete Committee. The recommendations and conclusions which it contains are based on experimental data, obtained in 1904–1921 at the Stuttgart Technical School. The customary formula for calculating the compressive strength of a reinforced concrete beam gives results which are too high, as other factors must be considered. These include (i) the bending effect (which diminishes with the depth of the beam), (ii) the number of loads (as several loads have a smaller bending effect than a single load of the same weight), (iii) the transverse section of the beam, as in T-beams the ratio of bending pressure to compressive strength is lower than with plain beams, (iv) the richness of the concrete, and the nature of the aggregate, and (v) the atmosphere in which the concrete is cured (a moist atmosphere producing slightly stronger beams).

The T-beams cracked most seriously in the base of the vertical portion during bending tests, the cracks of the horizontal portion of the beam commencing at each end and slowly extending to the centre. The provision of bent reinforcing bars near the ends of the beam diminished, but did not prevent, the end-cracks. The best results were obtained with T-beams, in which the ¼-in. reinforcing rods were placed 4 in. apart in the horizontal part of the beam, the vertical part of the beam being 8 in. deep and reinforced by four longitudinal rods. The crushing strengths and the ratios of modulus of rupture to crushing strength in various beams tested are shown in three tables. The work is an excellent summary of the tests on beams, made under specially careful conditions. Unfortunately, the details are only available to those readers who are sufficiently familiar with the German language.

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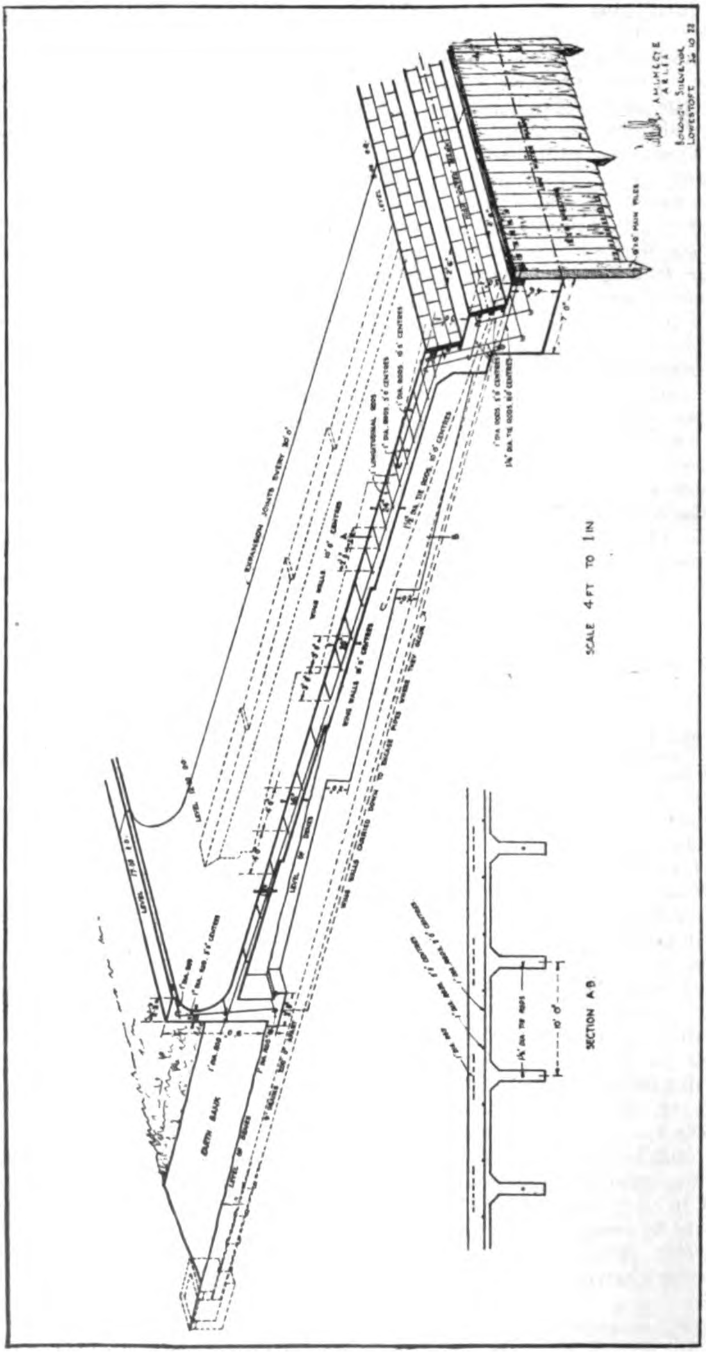
* *Widerstandsfähigkeit der Druckzone von Eisenbetonkörpern.*"



Mr. S. W. Mobbs, A.R.I.B.A., Borough Surveyor.

NEW SEA WALL, NORTH DENES, LOWESTOFT. (See p. 405.)

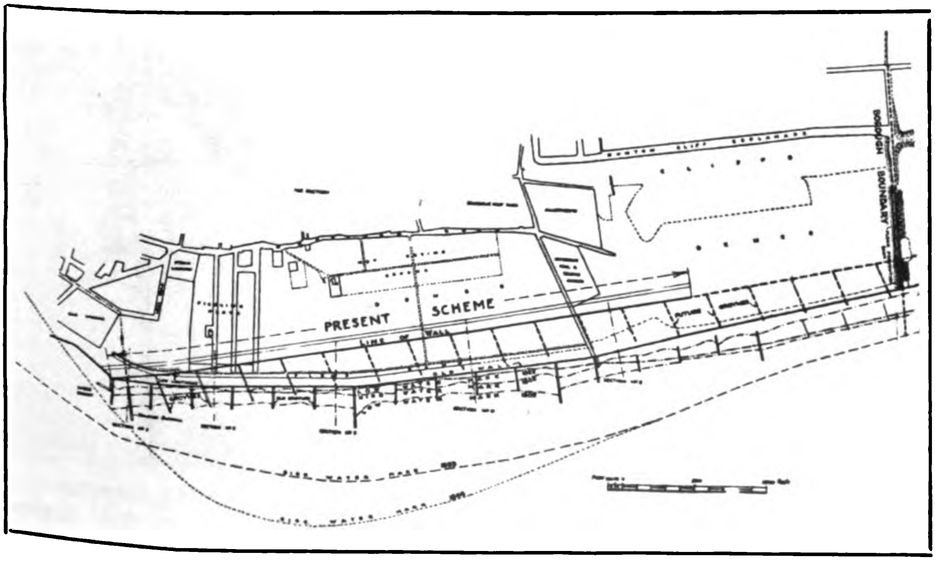
NEW SEA WALL, NORTH DENES; LOWESTOFT.

Many years ago the beach of the northern part of Lowestoft was on Whapload-road, and High-Water-Mark was the bottom of the cliffs along the west side of Whapload-road. The Denes consist mainly of blown sand with layers of shingle deposited by the sea as it receded eastwards till about the year 1840, but from that time onwards there was a tendency for the sea to encroach again. In the year 1844 High-Water-Mark was 970 ft. eastwards of the existing sea wall opposite the Rectory shown on the plan, while in 1883 the sea had come into within 650 ft. of the present sea wall.

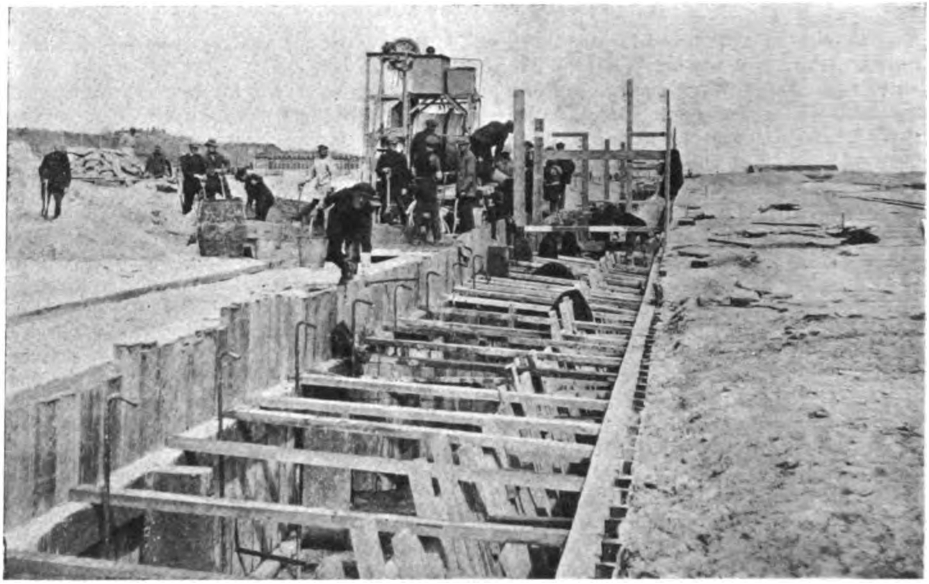
A sea wall was designed by the late W. T. Douglas and constructed in 1901, and a number of groynes were put in to hold up the beach. In 1912 High-Water-Mark had reached within a few feet of the wall; at the present time High-Water-Mark is at the sea wall, and Low-Water-Mark, which in 1912 was 150 ft. from the sea wall, is now within a few feet of the wall. During a scour last November Low-Water-Mark coincided with High-Water-Mark immediately at the sea wall, and the beach for about half a mile was non-existent. North of the swimming pool the depletion of the beach has been much more gradual, and High-Water-Mark is about 250 ft. further westward than in 1844.

There are two factors accounting for the facility with which the beach is removed during easterly winds. One of these is the fact that the whole of the land is, as before stated, deposited material consisting of shingle and sand which is easily eroded, and there is no substantial bottom there for a good many feet down. The other reason is that the coast line in this vicinity forms a salient causing it to be continually exposed, and also causing the tide to off-set between the swimming pool and the Ness Point; thus the natural flow or feed of shingle is diverted past the groynes instead of being arrested by them.

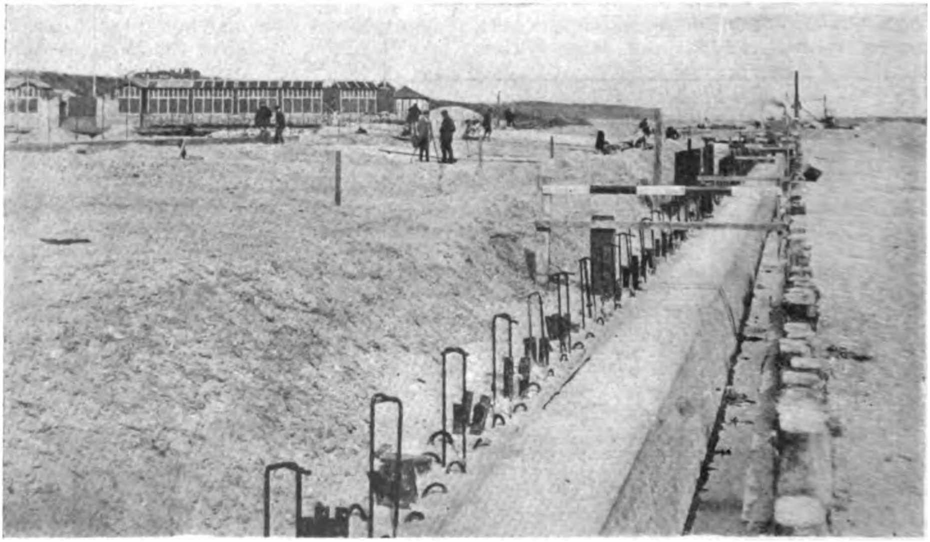
The Douglas Wall showed signs of giving out due to undermining about 1912, and during 1915 and 1916 destruction of this wall commenced, and the groynes in front, taking increasing strain, rapidly wore out. A long length of the Douglas Wall has been replaced by heavier and deeper walls between the Low Light and just south of the swimming pool road, but owing to the continual deepening of the sea opposite this point these walls, although still intact, are beginnng to follow the Douglas Wall



New Sea Wall, North Denes, Lowestoft.



Shuttering in Position for First Set-back.



Showing Completion of First Set-back and Reinforcement for Flood Bank Wall in Background.

NEW SEA WALL AT LOWESTOFT.

into the sea. During the last winter a long line of the existing sea wall started to move eastward, tending to slip down into deep water, and this was only checked for the time by an enormous expenditure in temporary works.

Realising the economic impossibility of maintaining the old works, the Corporation decided to erect a new wall in a straight line from the Ness Point (see plan) which it is essential to maintain owing to the position of the sewer outfall past the back of the Low Light running in a line just east of the swimming pool a distance of 4,300 ft. This will straighten out the existing salient in the coast line, and should, when completed, relieve the Corporation of any cost in the way of maintenance or groyning for a number of years, except at the south where it will be necessary to put in four or five groynes at once.

The scheme was designed by Mr. S. W. Mobbs, M.Inst. M. & Cy. E., A.R.I.B.A., Borough Surveyor. The wall consists of a very heavy low-stepped reinforced concrete sea wall protected by sheet piling on the seaward side and having a slope of 1 in 20 at the back; it is 60 ft. wide, terminating with a reinforced concrete flood wall with a turf bank supporting the rear, as shown on the drawing. It is intended to groyne the whole length of the new wall as soon as circumstances permit.

The work is being carried out by direct labour, thus absorbing 400 of the local unemployed, and, having regard to the magnitude of the scheme, up to date machinery has been employed embodying Ransome's petrol-driven concrete mixers, "Erie" steam shovel, and Gwynne's "Invincible" pump, which are a necessity in keeping costs to the lowest possible limit.

The design has received the approval of the Minister of Health, and the cost, which is estimated at £100,000, is being partly borne by the Government through the Unemployment Grants Committee and partly by the local rates.

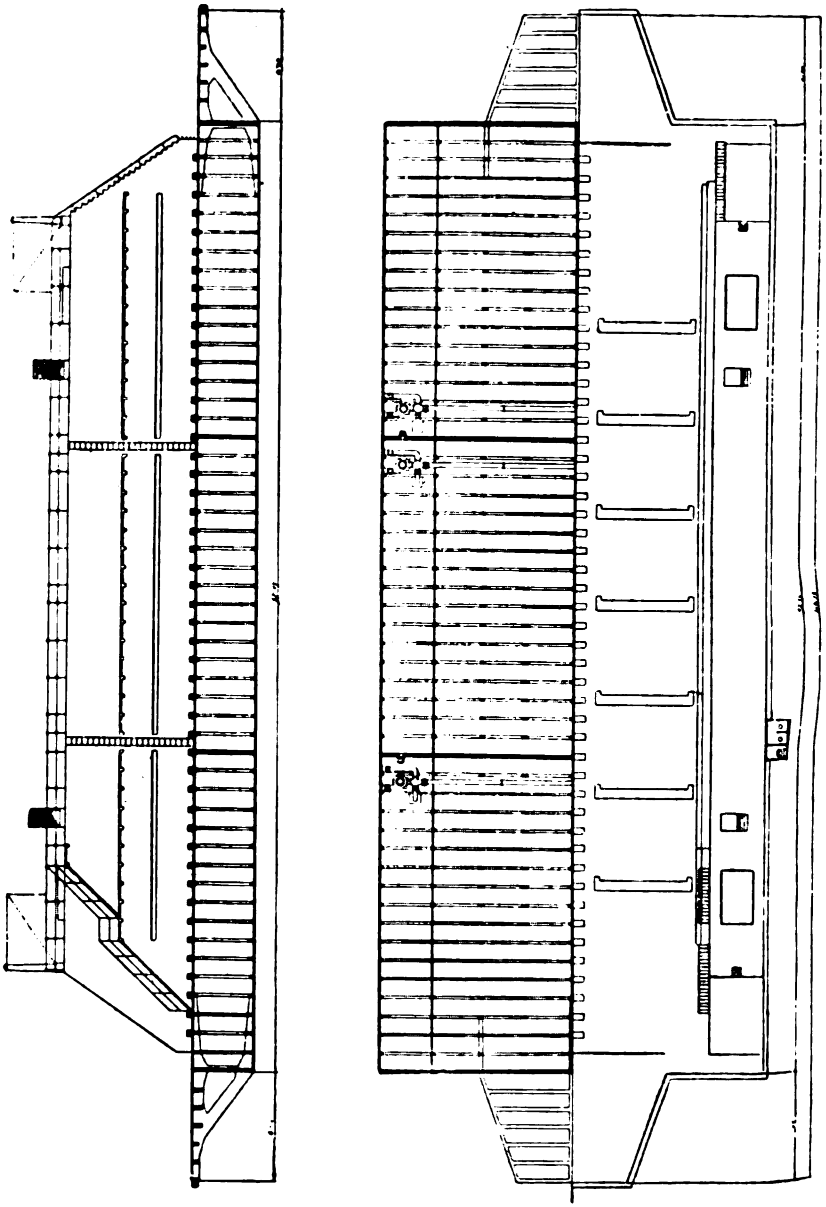
"Concrete Roads."

Owing to the increasing interest in the subject, due to a realisation that a great improvement in road construction is needed to keep pace with modern traffic requirements, the first edition of "Concrete Roads" is now out of print. A second edition, enlarged and revised to include the latest practice, will be ready about the end of June. The price of the second edition will be reduced to 5*s*.

Referring to the progress of the all-concrete road in America, the late Lord Northcliffe, in "My Journey Round the World," says:—"I have been thinking over my Canadian and American excursions . . . Wherever you go in the United States and Canada gangs of men are making great new concrete roads for the accommodation of heavy and light motor traffic. We are very slow in this matter."

Repairing Concrete Roads.

In the course of a recent address, Mr. Leroy C. Smith, Engineer of the Wayne County Road Commissioners, dealt with the question of repairing all-concrete roads. In cases where the concrete pavement becomes shattered or is broken down in an area of at least 1 square yard, he said it was imperative that it should be repaired by some other method than the application of bituminous material and aggregate. Such holes must be torn out, and it was the best practice to dispose of the broken concrete, not using any of it in making the patch. It was desirable to enlarge the hole in area to a point where but few cracks radiated from or circled about the hole. The excavation should be made at least 1 in. below the old concrete; the ragged edge should be as nearly vertical as possible, if anything, it should be undercut so that the patch would give support to the surrounding concrete. No bituminous treatment should be given to the edges of the old concrete. He also urged that small quantities of tar or asphalte should be applied to any joints or cracks three or four times a year. Discussing the cost of keeping all-concrete roads in repair, he said he knew of no all-concrete roads which cost more than from 2 to 4 per cent. per annum in maintenance costs.



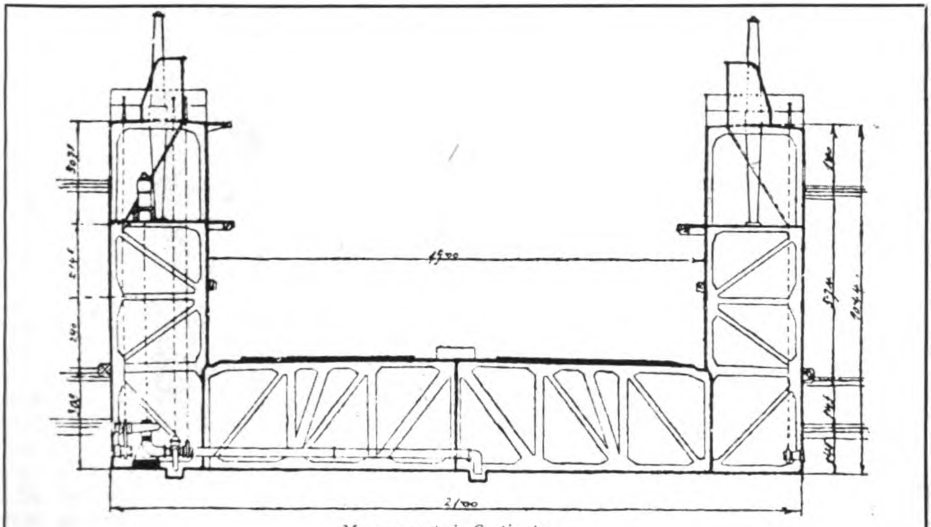
Measurements in metres.

Fig. 1.—An Italian Floating Concrete Dock: Longitudinal Section and Plan. (See p. 409.)

AN ITALIAN FLOATING DOCK.

THE first floating dock constructed of reinforced concrete in Italy has recently been launched at Trieste. It was designed for use in the Adriatic, and its principal dimensions are :—Total length, 214 ft. ; length above water, 174 ft. ; inside width, 50 ft. ; total width, 70 ft. ; inside height, 35 ft. ; total height, 57 ft.

The general structure is shown in *Fig.* 1, whilst *Fig.* 2 shows the arrangement of the ribs along the two main walls and the intervening passage-way. The details of the reinforcement are in accordance with the usual standards. Attention may be called, however, to the use of a well-trowelled 1 : 3 cement mortar, rather more than ½ in. thick, being regarded as sufficiently impermeable to water. For the construction, 320 tons of Portland cement, 1,280 cu. yds. of aggregate and 200 tons of mild steel reinforcement were employed. The total weight of the structure, including plant and machinery, is about 1,700 tons.—*Il Cemento.*



Measurements in Centimetres.

FIG. 2. AN ITALIAN FLOATING CONCRETE DOCK : TRANSVERSE SECTION.

Unemployment Relief Schemes.

IN a circular to local authorities, the Ministry of Health states that the Government has decided that, as regards all works in respect of which grants are given expressly because of unemployment by the Unemployment Grants Committee or by any Government department, it shall be a condition of the grant that all contracts for or incidental to the works are to be placed in this country. This follows a similar condition with regard to schemes in respect of which guarantees are given under the Trade Facilities Act. The Government also urge that, in view of present conditions, the same principle should be adopted, in the absence of special circumstances, in the case of all contracts in respect of works carried out by local authorities, whether or not the expenditure on the works is assisted by grants from the national exchequer or from the Road Fund.

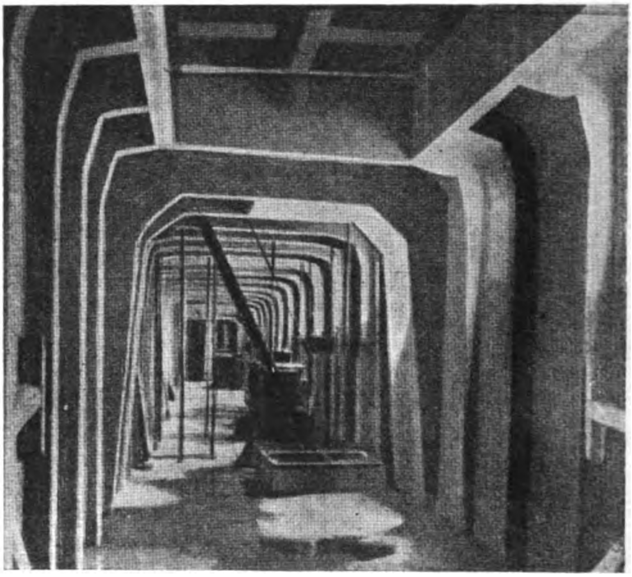


FIG. 3. PASSAGEWAY.

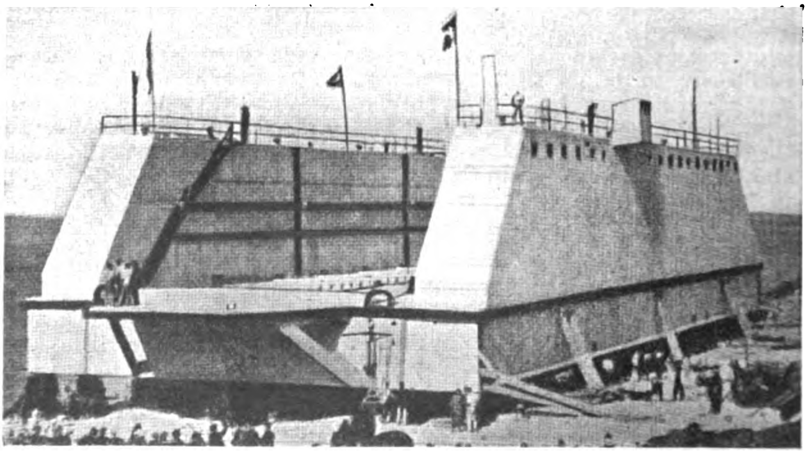


FIG. 4. JUST PRIOR TO LAUNCHING.

AN ITALIAN FLOATING CONCRETE DOCK. (*See p.* 409.)

AN UNUSUAL FIRE TEST.

A FIRE which occurred in January this year at Duluth presented some interesting features, for two reasons: (1) the building was in course of construction and the shuttering had not all been removed, and (2) the concrete had been laid at temperatures between 20 deg. above zero and 20 deg. below zero. The building is a 6-story structure, 128 ft. wide above the third floor, and 172 ft. wide below, extending some 490 ft. into a lake. At the time of the fire all the fifth floor was poured and some 108 ft. of the sixth floor at the south (Lake Superior end). Form work and reinforcing steel were in place for another large section of the sixth floor, and form work for 108 ft. of roof.

In a description of the fire and effects given before the American Concrete Institute, Mr. A. R. Lord said the fourth and fifth stories of the building were enclosed in canvas and heated by salamanders which had been charged with anthracite a short time before the discovery of the fire. An explanation offered for the rapid spread of the fire is that the upper part of the fourth story was heavily charged with gas from the salamanders and spread the flames throughout the length of the building. The form work in the sixth, fifth and fourth stories burned practically without check and was substantially consumed. In general, the column and drop forms had been removed and the surface of these members received the full heat of the fire. Many of the posts fell, and some of the joists either fell or were supported precariously on one or two charred posts; the form work offered little by way of support to the slab.

The fire, starting in the fourth story and, involving that entire story within a few minutes, spread rapidly to the fifth and sixth stories above. Form work was in place for the sixth floor at the north end, and the reinforcing steel was practically ready for the concrete. All this work was completely consumed and the steel dropped and tangled. The spirals and column verticals were largely protected by the metal forms, but the light wire spirals of this story were so badly twisted and deformed as to be almost useless. At the south end the sixth floor had been placed five days before and its supports were entirely consumed, as was also the form work for the roof. The slab came through this severe test with little more than surface damage.

The principal damage to the building was due to the presence in the aggregate of large quantities of granite pebbles which fractured when subjected to sudden intense change in temperature. Everywhere that spalling occurred the surface presented heat-fractured pebbles. Where the form work was entirely consumed the surface of columns and drops, which were exposed for the entire length of the fire, was composed largely of such fractured pebbles. The quartz present in these pebbles was so finely divided as to be scarcely recognizable, and the presence of iron further disguised the material. Where the form work was in place the fire ate its way to the concrete at the cracks between boards or directly over the joists where the extra material made a hotter fire and in such places incipient spalling, commonly very shallow, was general.

In a fire of this limited duration the square columns stood as well as the round; in both shapes the damage was a surface damage almost entirely. The deepest spalling uncovered some of the slab reinforcing steel, principally in the direct bands which were from one-half to one inch above the forms. The amount of spalling indicates that temperatures considerably in excess of 1,070° F. (at which quartz fractures) existed over large areas in this fire.

Some of the panels poured the same day fell when the forms were removed and some actually stood up. Out of 16 panels placed that morning 12 fell. All the columns remained standing except two, and all the exterior columns were inclined slightly inward and have been removed and replaced. The flat slab of the fourth floor directly below came through without injury; this floor was five days old. Some fourth-story columns on the west face were cracked by the early removal of the slab forms by the fire. Careful examination failed to show any cracks in the slabs in any part of the building outside the small area that was poured that day.

In the construction of the building the materials, both gravel and sand, were heated by means of steam jets in the piles and also in the bins over the mixer, keeping them free from snow and ice and finally raising them to a temperature of 70° F. Concrete was spouted to a hopper and poured to its final position in the forms. The top surface of the freshly-deposited concrete was not protected and presumably froze, but as soon as men could walk on it a layer of sawdust and shaving some 4 in. thick was spread over it. In half an hour or less the slab was warmed through again, and the top became decidedly warm. The floor finish was not placed with the slab, but is to be placed after the building is fully enclosed and heated. " While this method departs from usual practice in several respects," says the author, " the ' acid test ' of fire seems to have demonstrated its effectiveness under extreme conditions. Workmen removing the concrete which fell and that had been heated only for a part of one day found it very hard to break with bull-points and sledges."

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

***Readers are cordially invited to send in questions relating to concrete, which will, as far as possible, be answered direct and subsequently published where they are of general interest. We cannot give complete designs; these are the province of the consulting engineer. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.*—ED.**

CURVED BEAMS.

QUESTION.—Would you explain the manner in which a curved beam, as in the floor of a circular water-tank, should be designed ? Also, what is the most up-to-date method of determining the dimensions of a tee-beam when designing ?—T. W.

ANSWER.—In general, a beam curved in plan is subject to twisting moments as well as bending moments. The subject is a complicated one, and there is little experimental information on beams subject to twisting. It is therefore desirable that all such beams should be of ample proportions, and amply reinforced. In the case given, the sides of the tank would also act as beams. The calculations of a T-beam in general cases have been given so frequently in this journal, and is so large a subject, that we refer our correspondent to previous articles.

DISTRIBUTION BARS.

QUESTION.—In designing slabs both Mr. Andrews and Mr. Cantell (Part I) give tension bars running both ways of the slab, but do not seem to mention distribution bars. Is not the usual practice to have tension bars only parallel to the short side and distribution bars along the top parallel to the long side of the slab ? Would the distribution bars take the place of the tension bars running parallel to the long side of the slab ? How are distribution bars calculated ?—F. J. S.

ANSWER.—The object of " distribution bars " is to prevent cracks in the direction of the short span due to shrinkage of concrete, temperature variations, etc., and to ensure point loads calling a greater width of slab into play than would otherwise be the case. We recommend using the same size rods as the main reinforcement but at four times the spacing (see *Reinforced Concrete Simply Explained*, p. 22). Distribution rods are generally placed over the main reinforcement but not near the top of the slab. If tension bars are already provided running both ways, nothing further appears to be required.

FLAT SLAB CONSTRUCTION.

QUESTION.—Will you advise me which book I should get on the flat slab type of construction extensively used in America ? —M. A.

ANSWER.—*Concrete Plain and Reinforced*, by Taylor & Thompson (London : Chapman & Hall, Ltd., price 30*s*. net) deals very well with this subject.

REINFORCED CONCRETE ESTATE ROAD.

By DEL TONKIN.

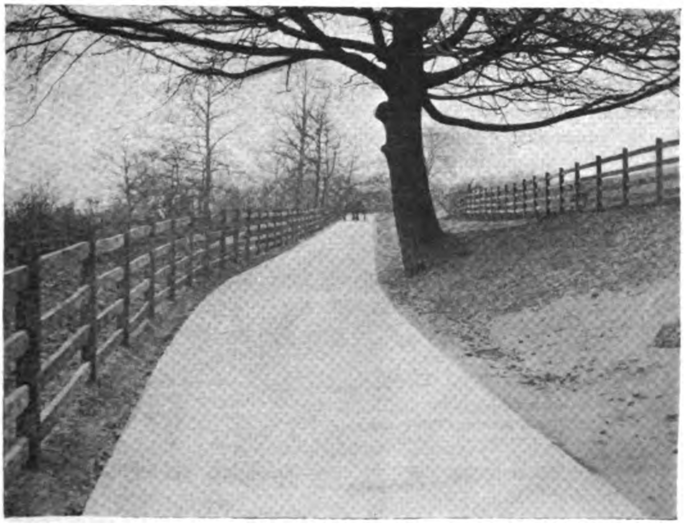
THE rapid progress of reinforced concrete road construction is shown by the accompanying photographs, illustrating a reinforced concrete road through the private estate of Mr. Ernest B. Hall, of Hales Hall, Market Drayton, Salop.

The road is 2½ miles in length, 9 ft. wide, with passing places 40 ft. by 9 ft., and apron gateways 27 ft. by 9 ft. 6 in. extra to the 9 ft. road.

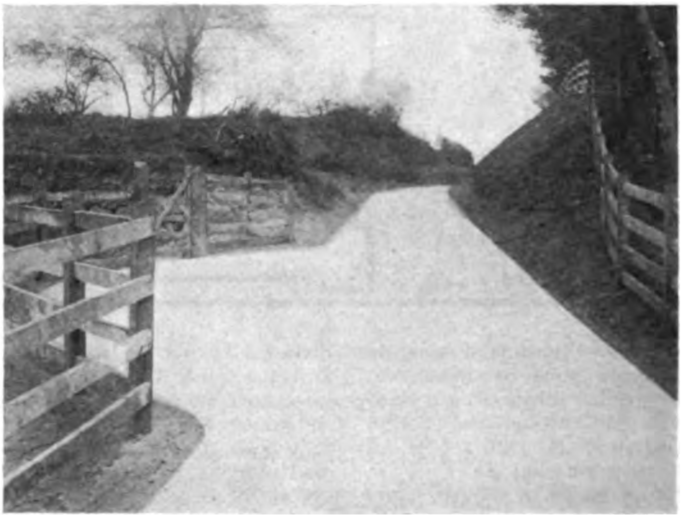
Commencing at the village of Hales, where it links with the public main road, it winds through woods, pasture, and plough land, serving four different farms, and is a great boon to the farmers. Prior to the construction of the road the farmers were compelled, owing to the heavy and broken nature of the existing cart-tracks, which were the only roads through the estate, to go an unnecessary distance to get to the town of Market Drayton with their produce, and it is estimated that not only is the distance shortened by 40 minutes, but where it took two horses to draw a reasonable load over the heavy and broken ground the same load can easily be taken over the concrete road with one horse.

The concrete road was commenced in August, 1922, and completed in February, 1923. Being a wet summer, progress was slow, there being no concreting during wet weather, or in the winter months when there was frost. During this period, the frost and rain-protecting frames (illustrated in this Journal for December 1922) were used with success, no frost penetrating the concrete.

The concrete is two-course work, the bottom course 4½ in., the top course



REINFORCED CONCRETE ESTATE ROAD AT HALES HALL, MARKET DRAYTON.



REINFORCED CONCRETE ESTATE ROAD AT HALES HALL, MARKET DRAYTON.

1½ in. thick. The proportioning of aggregates is 4 : 2 : 1 for the bottom course, and 3 : 1½ : 1 for the top course. A gauge banker was constructed for measuring the quantities, and the proportions strictly adhered to. The reinforcement was laid 1½ in. from the underside of the bottom course. The gravel for the bottom course was taken from a pit on the estate, but before being used it was screened and washed; the sand for the bottom coat was also obtained locally. The top wearing coat is of granite chippings, graded ⅝ in. to ⅛ in. The surface was finished with wood screeds, so made as to leave slight corrugations. The cross-fall is generally 1½ in., but there is no camber on gradients.

The cement used was the "Dragon" Brand of the South Wales Cement Co., which I have no hesitation in recommending for road construction.

Each batch of concrete was mixed mechanically in a drum-type mixer. The reinforcement used throughout the whole of the road was "B.R.C." fabric. The simplicity of laying this fabric expedites the progress of work, as it arrives on the site ready for immediate use.

A section of the road is treated with "Novoid" for experimental purposes, for a length of 260 ft., divided into two equal sections of 130 ft. for the top coat only, one section being 4 : 2 : 1 with "Novoid"; the other 3 : 1½ : 1 with "Novoid."

The work has been carried out by Mr. Ernest Ireland, Contractor, of Bath.

INTERNATIONAL ROAD CONGRESS AT SEVILLE.

As was to have been expected, the newest form of road construction—the all-concrete roads—was the subject of much attention and discussion at the Fourth Congress of the Permanent International Association of Road Congresses held at Seville last month. Seventeen papers were devoted exclusively to the all-concrete road, by representatives from England, the United States, Canada, France, Italy, Belgium, Holland and Spain.

Our immediate interest in this Conference was naturally the use of concrete in road construction, and it appeared that the more information the delegates obtained the more enthusiastic they became. The Americans are naturally ahead of us in this subject in view of their longer experience, and their papers were of very great value as giving the knowledge gained not only by experiment but also by practice. The English papers showed that experimental work has been carried out very thoroughly in this country, and that the practical stage had now been reached. The delegates were agreed that for the heavy traffic carried by present day rubber-tyred vehicles the concrete road is admirably suited, and its non-slipperiness makes it additionally attractive as the horse is thereby catered for, an important point in agricultural and Scottish districts. France and Belgium in particular have been carrying out experiments with special cements, but as their traffic conditions have not yet reached the density and weight with which we have to contend one cannot express a definite opinion on them. It is suggested, however, by the French that for roads on which the loads are not heavy enough to justify concrete, a method which they have employed, namely, reballasting with macadam and, during or afterwards, incorporating an aggregating matter, formed by a mixture of sand and hydraulic binding might be found suitable. The extra cost of the cement is claimed to be more than compensated for by the increase of life of the road.

The use of concrete in tramway construction was also emphasised. During the discussions there appeared to be a feeling that the fast light tramcar would eventually triumph over the heavy cumbersome vehicle and would probably prove more economical.

The following extract from *The Times* summarises the feeling of the Conference with regard to concrete roads:—

"What is being done to meet the urgent needs of the situation has been indicated by the discussions at the Seville Congress. Many types of road are being made the subject of experiment. In the United States, where, according to the last Census, the existing motor vehicles number eleven millions, the concrete road has found most favour. Over sixty million square yards of this type of road have been constructed and have shown a capacity to withstand modern traffic conditions. In Great Britain experience of the concrete road is more limited, but this method of construction is being employed for some of the new arterial highways and in centres of heavy traffic, and is stated to have given results which justify a further extensive trial."

The social side of the Conference was generously catered for, and many very enjoyable entertainments were given.

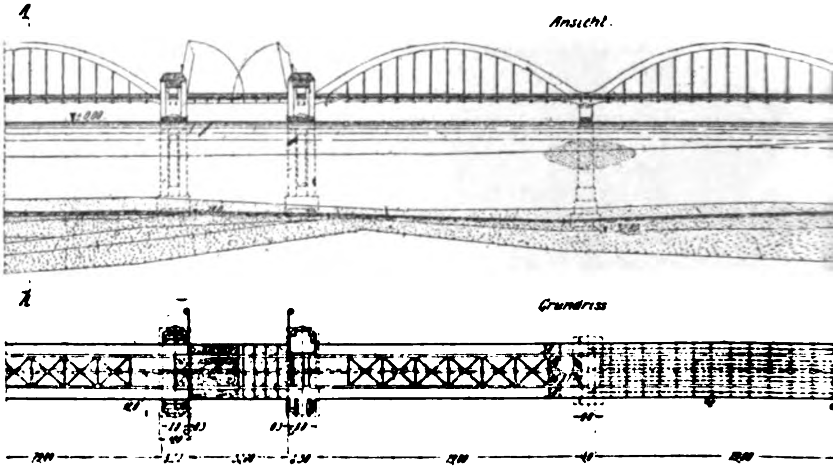
Abstracts of the papers on all-concrete roads are given herewith:—

ENGLAND.

Mr. Alfred Dryland, M.Inst.C.E. (County Engineer of Middlesex), in the preparation of his paper, obtained the opinions of ten road engineers in this country who have laid all-concrete roads, and embodied the experience gained by these engineers with his own in his address.

Briefly, the practice recommended by Mr. Dryland and those he consulted is: Aggregate may consist of any stone hard enough to resist the wear and tear of traffic; it must be clean and graded to reduce voids to a minimum. Sand should be washed and not contain a large percentage of fine grains. Machines for finishing the surface permit the use of drier concrete, and also result in more correct camber. For single-course roads the concrete should not be leaner than 1 : 2 : 4; for two-course roads the proportions should be 1 : 1 : 6 for the base

[*Continued on p.* 418.]



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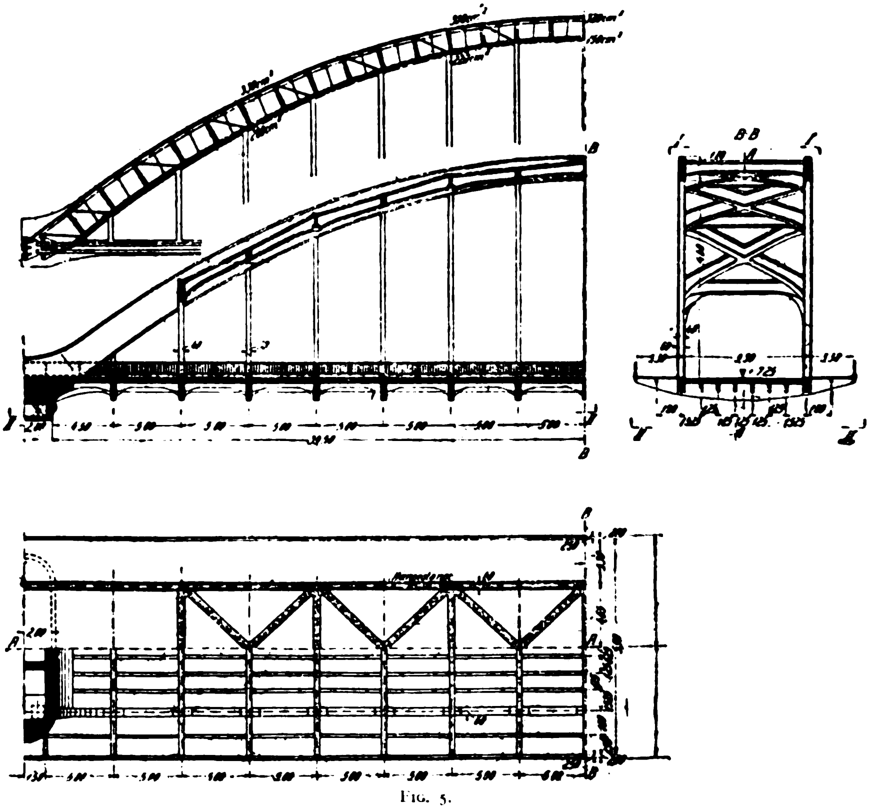
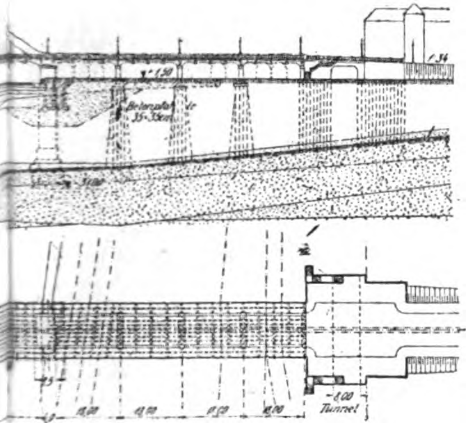


Fig. 5.



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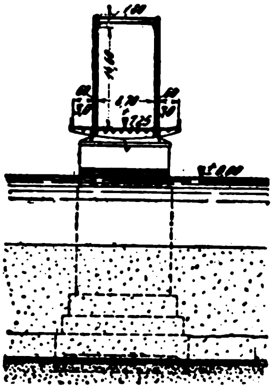


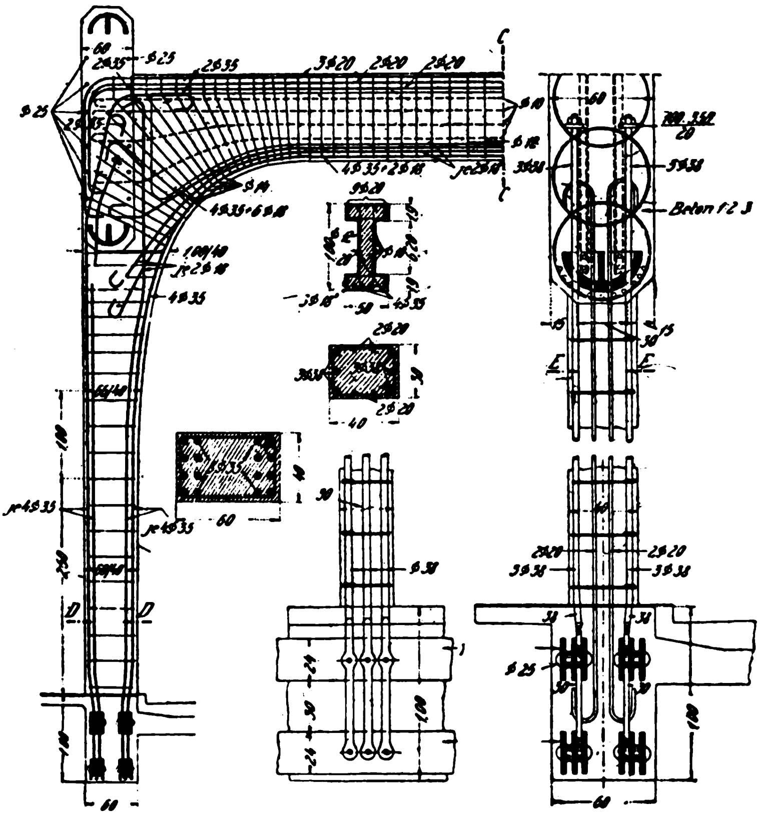


Fig. 6.

ETITION FOR THE LIMFJORD BRIDGE IN DENMARK. (*See p.* 383.)

INTERNATIONAL ROAD CONGRESS AT SEVILLE.

[*Continued from p.* 415.]

and 1 : 1 : 3 for the surface course. The single-course road is more homogeneous and results in more uniform expansion and contraction. As there are only a few places where the stability of the foundation is sufficient to carry the roadway and traffic, the use of reinforcement is recommended. The foundation should be finished to the same camber as the top of the road, and should be thoroughly consolidated and drained. If the subsoil is clayey it is necessary to interpose a layer of at least 2 in. of fine broken stone.

The concrete should be at least 6 in. thick (depending on the traffic); for heavy traffic it should be 8 in. thick if reinforced or 10 in. if unreinforced. If possible the whole width of the road should be carried out in one operation.

The principal advantages of the concrete road are: Stability (the greater part of the other road surfacings get deformed by the wear of their constitutent elements by the traffic); concrete surfacings offer less resistance to traction than the other more flexible ones; for new roads this process is not much dearer than other improved systems, and if the traffic is fairly intense, with vehicles with rubber tyres, it will resist for a number of years at insignificant expense for maintenance, and it will always serve as a foundation for another surfacing of concrete or of bitumen.

THE UNITED STATES.

Ten papers by United States authors were presented together.

Mr. E. C. Blosser (State Highway Engineer of Ohio) recalled that the first all-concrete road was built in America in 1892, and was, he said, still in good condition in spite of the heavy traffic it carried. In Wayne County no less than 62,000,000 sq. yards of all-concrete road had been laid since 1907. The reason for the popularity of the concrete road was that it did not require special materials or special processes for its construction.

The following advantages of concrete surfaces were put forward by Mr. H. J. Knelling (Construction Engineer to the Wisconsin Highway Commission): (1) The materials do not require special technical knowledge to prepare; (2) Concrete is but little affected by heat and cold, rain or sun, and can therefore be used in all climates; (3) Materials are generally found near at hand; (4) The surface is not slippery, except in frosty weather, and does not hold rain; (5) Cost of maintenance is very low, and the filling of joints or cracks when necessary does not unduly interfere with traffic; (6) The road is easily discernible at night; (7) After many years of use concrete roads can be used as foundations for other kinds of surfacing.

Mr. H. S. Mattimore (Engineer of Tests, Pennsylvania Highways Department) made the following points in connection with the materials to be used:

Fine sand is not to be recommended. Sand should be free from slime, clay and organic matter. Ninety per cent. at least of the sand should pass a ¼-inch sieve; not more than 30 per cent. through a No. 50 sieve; and not more than 10 per cent. through a No. 100 sieve. Broken stone is generally preferable to round stone for the coarse aggregate; blast furnace slag may also be used; abrasion and resistance tests should be made.

Professor Duff Abrams dealt with the question of obtaining a good concrete by strict attention to the proportioning, the water content and supervision, and came to the conclusion that good concrete could be obtained with inferior aggregates if they are scientifically used.

Mr. Roy W. Crum (Engineer of Materials and Tests, Iowa Highway Commission) urged the importance of varying the proportions of cement, sand, and coarse aggregate to suit different conditions, and deprecated the tendency to adopt a 4 : 2 : 1 or any other mixture irrespective of the materials used.

In dealing with the finishing of the concrete surface, Mr. Clifford Olde (Chief Highway Engineer, Illinois Dept. of Public Works) pointed out that concrete which is spread, compressed, and smoothed by hand must be more plastic than when these operations are mechanically performed. The consistency can be determined by the slump test. Care must be taken that all the mixtures are of the same consistency so as to avoid undulations of the surface.

The concrete should be spread in a thicker layer than is actually required. If the work is done by hand, it is smoothed with a straightedge, leaving an excess of thickness of half an inch ; the ramming is performed with a wooden block having the same shape as the surface of the road with handles at the extremities. Then a small roller is passed over worked with a long swinging shaft and a rope. The surface is smoothed with a canvas 8 ins. wide and a little longer than the width of the foundation. The operation is repeated until the concrete commences to harden.

In summer the concrete should be covered with canvas. The next day the canvas should be taken off and a layer of earth spread 2 in. thick and kept moist for a fortnight. If there is danger of frost the surface should be protected with a layer of earth or straw, and, placing canvas at the sides.

In Illinois some experiments were made for protecting concrete during the summer. Chloride of calcium in grains was spread in the proportion of 3 lb. per sq. yd. These experiments showed that the chloride of calcium, which is dissolved in the water employed for the concrete in the proportion of 3 per cent. of the weight of the cement, probably accelerates the setting when the temperature is below zero and prevents it freezing.

The subject of expansion joints was dealt with by Mr. C. R. Ege (Manager, Highways Bureau of the Portland Cement Association).

The phenomenon of expansion having, he said, been known for a long time, endeavours were made to abolish the prejudicial effects by placing transverse joints at short distances, in some cases 25 ft. The phenomenon of contraction is another reason in favour of the use of joints. It has been found that for want of these joints transverse cracks appear at regular intervals and joints are placed to hinder the effects of contraction. Joints at regular intervals of less than 50 ft. undeniably reduce the number of transverse cracks.

In Illinois, during 1921, phenomena of expansion were observed, and as the joints were not prepared to prevent them the facing rose, as a general rule at a joint, in certain cases as much as 22 in., in other cases the surfacing broke. This phenomenon was produced once in about every 50 miles. To repair the damage the road was cut until the concrete could get itself back into place. When joints are placed, damages do not happen so frequently. The joints are placed at changes of level or of direction, at tangents, and always less than 200 ft. apart.

In the State of Washington, where heavy rains occur, expansion joints are placed at intervals of less than 30 ft.

When there are no joints, the only interruptions to the continuity are those produced by the stoppage of work, which are from 200 to 600 ft. apart according to the capacity of the concrete-mixers and the rapidity of working. At the end of the day's work, a plank is placed at right angles to the axis of the road and flush with it, and removed when work is renewed. Often short steel bars are placed embedded in the concrete to unite the two sections in order to avoid a difference of level between them. There are hundreds of miles of road built according to this process, and except for the expansion effects already mentioned they have given satisfactory results.

Mr. B. H. Wait (District Engineer, Portland Cement Association, New York), generally agreed with the previous papers, and mentioned that completely reinforced surfaces could be made by the circumferential method with wire netting in the upper part by placing grillages in both the upper and lower parts. As a matter of fact the necessary resistance of the surface can be obtained with concrete reinforced or not.

CANADA.

Mr. Gordon Grant (Chief Engineer, Highway Branch of the Department of Railways and Canals, Ottawa) dealt with the subject from many aspects : Coarse aggregate and sand must both be quite clean. Flat pieces and needles of stone must be rejected. Particles of feldspar and mica must be eliminated from the sand. Coarse aggregate should pass through 1½ in. to 2-inch rings, but not through a ¼-in. ring. Road stones must not be used unless their size varies so that there will be no voids. The proportions generally used in Canada are 1 : 1½–2 : 3. Foundations must always be drained, if hard frosts are expected, or the concrete will rise during the thaw. In Canada the concrete roads vary from

14 ft. to 24 ft. in width; the thickness varies from 7½ in. to 8½ in. in the centre and 6 in. at the sides. Expansion joints are placed every 35 ft.; they are filled with tar, asphalte, bituminous or fibrous substances, and vary in width from ¼ in. to ½ in. It is important to keep the concrete at each side of the joint at the same level. Canada possesses 650 miles of all-concrete roads, which have given highly satisfactory results.

FRANCE.

An account of a number of experiments in France was given by MM. Varvier (Ingénieur en Chef des Ponts et Chaussées, Lyons) and Nicolas (Ingénieur des Ponts et Chaussées, Paris).

Following an experimental concrete road laid in 1905 near Lyons, and which successfully resisted very heavy traffic for nine years, trials were made at Villefranche and Lyons. Some of these trials were more or less in the nature of strengthening existing macadam roads with cement.

The following method has been found most satisfactory: The surface of the existing road is broken and moistened, and covered with broken stone, which is also watered and rolled with an 18-ton roller. This is allowed to become quite dry before the surface is laid. Then three-quarters of the cement mortar is spread in thin layers, thicker in the centre than at the sides so that the surfacing might be thicker in the centre, and each of these layers is rolled dry. The last of the layers is moderately watered, so that the mortar might penetrate well into the spaces between the stones. The remaining quarter is employed during the rolling at the necessary places. When the last layer of concrete is dry it is covered with a layer of sand 4 mm. thick. The object of the sand is to protect the surfacing during the first few days from the effects of traffic and to fill the voids produced by the setting of the mortar. The average thickness of the surfacing is 10 cm. The roller passed on an average 150 times. These surfacings have resisted well. On some sections they were coated with tar in September, 1921.

Borings effected showed that the mortar had filled all interstices—but in order to obtain a good result it is necessary: (1) That the stone be well cleaned; (2) Not to employ too much water; (3) To execute the junctions with great care, if the work is done in half widths, because they constitute weak points.

These surfacings when tarred constitute an important improvement as compared with ordinary macadam, and cost only about 20 per cent. more.

A second set of experiments was carried out at La Ferte and Chambery. After having reballasted in the ordinary way, the mortar was spread on the surface with brooms in quantities of 8 to 11 cu. dcms. per sq. metre of surfacing, of a thickness of 8 cm. In the first case the mortar was composed of Portland or slag cement and sand, in the proportion of 600 to 700 kg. of cement per cu. metre of sand, and in the second case of hydraulic lime in the proportion of 400 kg. per cu. metre of sand. The rolling was carried out on the whole width with a 15-ton roller.

Very different results were observed, according as traffic was forbidden during a month or permitted after 3 days. The higher cost as compared with ordinary macadam was about 11 per cent.

Other trials were made which consisted in spreading, after the rolling, Portland cement, Roman cement or hydraulic lime, in a very liquid state so that it might penetrate well to the bottom. A layer of sand was afterwards spread to a thickness of 1 cm., which was not rolled. The surfacing with lime did not give good resistance, while the Roman cement was less resisting than the Portland, the latter adhering better to the stone.

In the neighbourhood of Nancy mortar was laid between two layers of broken stone, the whole being rolled afterwards.

Experiments were also carried out with Teil lime concrete, and with a special cement called "Soliditit," but no definite conclusions were reached.

A trial carried out with the addition of "Rhoubenite" to the concrete showed that the surfaces in which this material was not used are in as good a condition as those in which it was used.

Another experiment was made with the "Pont-à-Mousson" system. This system consists in placing in dry concrete, 10 to 20 cm. thick, foundation bars embedded in rich Portland cement; the reinforcements, 4 to 5 cm. in height, are disposed in squares, with separations which vary according to the strength required. It is necessary to ram thoroughly the upper layer of concrete so as to fill all the voids without altering the position of the reinforcement. With a view to slow setting, the surface is watered twice a day and traffic is permitted 5 days after the work is completed. The trials are too recent and were carried out on too small a scale to allow of drawing any definite conclusions.

Some trials of concrete surfacings into which materials are set in the form of mosaic work have been made in the Department de la Vendée, according to the system of M. Guiet. The rolled roadway serves as foundation for a layer of concrete. The stone is laid by hand in the concrete so as to form mosaic work, or small paving stones of equal size are inserted like mosaic work. In all these cases the concrete is compressed with light tandem rollers of from 2 to 3 tons. Traffic can be permitted after 24 hours in summer and in winter after 2 or 3 days. The surfacing should be covered with damp sand for 8 days. This method is known by the name of the "Indeformable."

BELGIUM.

A paper presented by MM. Claeys, Goorieckx, and Mulhen (Ingénieurs Principaux des Ponts and Chaussées, Belgium) was chiefly devoted to an account of a material called "Rhoubenite."

The inventor of this material endeavoured principally to arrive at a better impermeability, and to achieve this end adds to the concrete very fine sawdust, chemically neutral but compressible, and with considerable powers of absorption, to which a tarred mixture is added.

It is claimed that the resistance to compression of "Rhoubenite" concrete is equal to that of ordinary concrete of the same proportions and that it does not provoke chemical reactions which would oppose the setting. When the "Rhoubenite" powder is incorporated with the concrete, and during the first period of setting, the tarred product remains impregnated in the sawdust and does not intervene in the setting. Later the tarred product, under the action of expansion and contraction, fills the pores produced by the evaporation of the water and the completion of the setting, and also acts as an elastic body in the concrete.

On the subject of expansion joints, the authors of this paper think a good result would be obtained by making provisional joints formed by a line of bricks or wooden blocks until the definite setting of the concrete, and then taking them away and filling the space with concrete or asphaltic mixtures.

ITALY.

MM. Luigi Luiggi (Inspector Superior del Cuerpo de Ingenieros Civiles) and Italo Vendone (Ingeniero del Instituto Experimental de Carreteras del Touring Club Italiano) in their paper referred exclusively to trials made of surfacings using a special aggregate, "Solidit it." This kind of surfacing was first used in Italy in 1911. The concrete was laid on an ordinary macadam foundation, previously smoothed and rolled. The width of the surfacing was 7·75 m. and the uniform thickness 7 cms. The work was carried out on half-widths. The gauging of the transverse section to ½ a millimetre approximately showed that during the first three years the average wear was 1–5 mm. and the maximum 8 mm. Later on cracks appeared and the surfacing was broken in several places. According to the opinion of the Chief Engineer of the Departmental Technical Bureau, the thickness of the surfacing should have been increased in order to obtain a longer life.

Another trial was made on the road from Milan to Monza, which gave excellent results during the three first years, but by reason of the exceptional traffic during the war and the impossibility of effecting repairs, as the traffic could not be deviated, the longitudinal joint became a deep rut, and the lack of homogeneity of the concrete, the trial having been made with different proportions, caused the formation of large and deep hollows, and it was necessary to abandon this surfacing. However, trials made with a normal wheel gave as result a wear of only 3·8 mm. after the passing of the wheel 16,000 times.

In the province of Milan more than 28,000 sq. metres have been laid with this material.

The general conclusion seems to be that "Solidit it" concrete surfaces, especially if the stone is very hard, offer great resistance to wear. However, traffic ruts are liable to be produced, and it is particularly unfavourable to execute this surfacing by half-widths. It may happen that surfacings which have resisted well during 2 or 3 years rapidly deteriorate afterwards in the case of roads with heavy traffic.

HOLLAND.

Mr. P. J. Van Voorst Vader, Jr., (Engineer of the Royal Water Department, Holland), described the trials on the highways between Amsterdam and Amersfoort. They were made upon a length of 150 metres and a width of 5 metres, divided into three sections of 50 metres.

On the first section, the surfacing was made of one single layer of 20 cm. thickness. The proportions of the concrete were 1 : 2 : 4.

On the second section the surfacing was made of two layers of a total thickness of 20 cm. The lower layer, 14 cm. thick, was made of poor concrete. The upper layer was 6 cm. thick and the proportions 1 : 2 : 4.

On the third section the surfacing was made of one layer of 15 cm. with a metal reinforcement. The composition of the concrete was 1 : 1 : 3. The section of reinforced concrete was made without expansion joints.

On the section with ordinary concrete expansion joints were set at various distances of 8, 9, 10, 11 and 12 metres. In the joints, sheets of cardboard impregnated with bitumen folded in two were placed. The concrete was made rather dry, with the result that the smoothing was difficult, so the upper layer was divided into two coats; the lower one of 4 cm. with little moisture, and the upper one 2 cm. rather more liquid. No cracks appeared, although the surfacing had to bear great differences of temperature. The expansion joints suffered damage to a certain extent.

The section made with lean concrete was destroyed rapidly.

The section made with two layers of concrete gave better results, but had to be replaced at the end of ten months. The section made with reinforced concrete offered better resistance; damage was noticeable only in the mortar, but the stones were not laid bare. As the section without reinforcement did not show any crevices, the difference of resistance must be ascribed to the richness of the concrete.

SWEDEN.

Mr. G. Dahlberg (Engineer, Djursholm, Sweden), in his paper dealt with the strains which concrete surfacing has to stand in case of swelling of the sub-soil, due to frosts.

After a summer during which the humidity of the ground decreased through evaporation abundant rains generally follow in the autumn, and the humidity of the sub-soil increases considerably. At the same time the temperature of the air falls and the frost begins. The ground freezes, and does so slowly on account of the heat produced by the congelation; this heat is transmitted to the ground underneath to a depth varying according to the specific heat, the conductivity of the soil, and the humidity of the air.

Experiments showed that concrete transmits the heat quicker than the broken stone or the sand, and much quicker than the cinders of coal and pounded peat. On the other hand, snow gives good protection against frost. The vibrations caused by vehicles prevent the water of the sub-soil under the surfacing from standing in a liquid state, and it will freeze quickly, whereas under the footpaths it is more distant from vibration and can remain in the same state. At the end of the winter it is noticed that the cracks increase a good deal; this may be explained by the greater dryness of the exterior air which provokes a more intense evaporation in the upper part of the sub-soil and therefore absorption of heat, permitting the frost to penetrate to a greater depth. The freezing of the ground produces an increase of volume, and this tends to make its way downwards where the resistance is less.

Other experiments demonstrated the increase of volume sustained by various substances through congelation, but experience has proved that the swelling of the ground may reach figures that will exceed those calculated when starting from a certain initial point of humidity, and the cause lies in the fact that other secondary phenomena are adding their action to that of the congelation. The cracks may show themselves not only in the centre of the surfacing but also at other points when there is much humidity in the sub-soil.

When spring makes itself felt the thaw of the ground is rapid, especially if there is snow on the footpaths, and when, through the action of various frosts and thaws, the ground contains a larger quantity of water than the average, some subsidings may appear.

In order to take precautions against damage from this cause, careful drainage is necessary, but as there are soils which retain the humidity in greater proportions than others it is well to place a layer of cinders, coal or gravel between the surfacing and the soil, which substances contain little humidity, and cannot consolidate themselves with the concrete under the action of frost. The thickness of this layer must exceed the maximum depth which the frost can attain. Another precaution consists in dividing the surfacing into two sections, making thereby a longitudinal joint.

Experience has shown that the most dangerous places are at the passage to the roadside. As far as possible the coating should be placed a little above the natural ground, in order not to touch the upper layer which generally contains substances of a great insulating power.

GENERAL CONCLUSIONS.

The following conclusions with regard to the all-concrete road were unanimously agreed to by the Congress, and passed in the form of resolutions :—

(I) Cement concrete carriage ways formed of materials of good quality in suitable proportions, built on sound and well-drained foundations, are capable, if carefully executed, of withstanding considerable traffic of heavy vehicles fitted with rubber tyres.

(II) Under the conditions and for the kind of traffic specified in the first conclusion, such roadways are stable in a variety of climates. They are smooth but not slippery, and the resistance they offer to tractive effort is slight. When worn, they can serve as a foundation for other types of surfacing. They are clean, give off little dust, and allow water to flow away readily. The necessary materials are to be found in nearly all countries. Visibility is good during the night, and there is but little noise. Vehicles running on these roadways suffer little damage. Against this, their repair in case of serious damage is difficult ; care is necessary in their execution which demands close and constant supervision ; traffic is necessarily interrupted for a considerable time. It is not easy to open trenches ; when cracks occur, the appearance is unsightly.

(III) Further investigations are required as to the effect upon road surfaces of traffic consisting of a large proportion of steel-tyred vehicles.

(IV) Materials for the concrete must be chosen with the greatest care. High grade cement and clean water should be used. The fine aggregate should be free from mud and organic matter, and only a small proportion of clay should be admitted ; an excessive proportion of fine particles should be avoided.

(V) It is of prime importance that the concrete should be as compact as possible. As a general indication based on previous practice one may quote the proportion in volume of 1 : 2 : 3½. In each case, trials should be made with the materials available so as to decide the proportion which will ensure a very compact concrete. If the surfacing is executed in two coats, the concrete for the lower layer may be poorer. The above proportions only apply to Portland cement.

(VI) The bed or foundation must be drained and consolidated with care. Drains must be laid below the level to which frost penetrates and at depths dictated by the nature of the soil. Where severe frosts are frequent it is well to amplify this precaution by placing beneath the surface a layer of non-conducting materials, such as coke, cinders, pulverised peat, etc.

(VII) To obtain favourable results the surface must be constantly and minutely inspected during progress.

(VIII) Wherever possible the surfacing should be executed for its full width at a single operation.

(IX) If the surfacing is in two layers the second layer must be placed in position before the setting of the first is complete and both layers rammed at one time.

(X) As regards the advisability of using mechanical plant, the question of economy should be considered in each case. It is, however, very important for the strength of the concrete to employ an appropriate and well regulated quantity of water. It is advisable that the levelling, tamping, and finishing be undertaken mechanically whenever possible. To ensure a homogeneous mixture it is well to utilise concrete mixers supplied with suitable devices for regulating the quantity of water and duration of preparation. The requisite proportion of water should form the subject of methodical study.

(XI) It does not appear advisable at present to express an opinion as to whether reinforced concrete should be employed in preference to mass concrete and in what circumstances. Studies and experiments should be continued upon this problem.

(XII) Researches should be continued as to the means to be adopted for reducing to a minimum the formation of cracks. The utility of providing expansion joints when the concrete is exposed to wide variations of temperature and humidity must be studied in each case. Every such joint, in effect, constitutes a source of weakness in the surfacing, and may prove the starting point of grave deterioration.

(XIII) In order to avoid accidents as the result of expansion caused by variations of temperature and humidity the introduction of expansion joints may be considered at distances to be determined in each particular case. The nature of the jointing materials and precautions to be adopted, both for this type of joint as well as for joints where work terminates, should continue to form the subject of experiments.

INSTITUTION OF STRUCTURAL ENGINEERS.

At a special general meeting of the Institution of Structural Engineers, the result of the election of the new Council, which comes into office on September 1, 1923, was declared as follows:—

President.—Major James Petrie, O.B.E.

Vice-Presidents.—Mr. H. J. Deane, M.Inst.C.E.; Dr. J. S. Owens, B.A., F.R.G.S., A.M.I.C.E.; Dr. Oscar Faber, O.B.E., A.M.Inst.C.E.; Sir Charles T. Ruthen, O.B.E., F.R.I.B.A.

Hon. Secretary.—Mr. M. E. Yeatman, M.Inst.C.E.

Hon. Editor.—Mr. H. Kempton Dyson.

Hon. Curator.—Mr. W. A. Green, M.A., B.Sc., A.M.Inst.C.E.

Hon. Librarian.—Mr. Ewart S. Andrews, B.Sc., A.M.Inst.C.E.

London Members of Council.—(Elected for three years): Prof. Henry Adams, M.Inst C.E.; Sir Henry Tanner, C.B., I.S.O., F.R.I.B.A.; and Mr. B. L. Hurst, M.Inst.C.E. Elected for two years: Messrs. D. B. Butler, A.M.Inst.C.E.; J. Mitchell Moncrieff, C.B.E., M.I.C.E.; and R. W. Vawdrey, B.A., A.M.I.C.E. Elected for one year: Messrs. P. L. Black, W. J. H. Leverton, and S. Bylander.

Country Members of Council.—Elected for three years: Mr. F. E. Wentworth-Sheilds, O.B.E., M.I.C.E. Elected for two years: Professor F. C. Lea, O.B.E., D.Sc., M.I.C.E. Elected for one year: Professor T. Hudson Beare, B.A., B.Sc., D.L., M.I.C.E.

London Associate on Council.—Elected for three years: Mr. J. T. Saunders, F.R.I.B.A.

Country Associate on Council.—Elected for three years: Mr. F. C. Webber.

London Associate-Member on Council.—Elected for three years: Mr. J. N. Peirce, A.M.I.C.E.

Country Associate-Member on Council.—Elected for three years: Mr. H. J. Collins, M.C., M.Sc., A.M.I.C.E.

CEMENT NOTES.

By Our Special Contributor.

The Setting of Portland Cement.

THE "quickening up" of cement, by which is meant the spontaneous change from a normal slow setting-time to a very quick setting-time, is a difficulty that has caused anxious moments to cement manufacturers and cement users. It has happened that consignments of cement have left cement works with normal setting properties and when put into use some weeks later have been found to set hard in a few minutes. No generally-accepted explanation of this change in setting properties has been presented, and although it is an occurrence that now rarely happens, the difficulty has been evaded rather than remedied.

When the reversion to quick setting has taken place it has usually been with a cement which had medium setting properties, viz., final set from thirty minutes to three hours, and the evasion of the difficulty has been helped by the deletion of the "medium setting" grade from the British Standard Specification for Cement. For general use, manufacturers now produce cement with a final set between three and six hours, and with the proper precautions in manufacture there is practically no risk of such a cement becoming quicker setting with time. The abandonment of the medium-setting grade of cement has been no disadvantage, because in the majority of concrete work the time of setting (so long as it is known) is of small importance compared with the speed of hardening, and this latter property appears to have no connection with setting properties as ascertained by specification tests.

It is, however, unsatisfactory that a problem of this nature should be left unsolved, and a pamphlet entitled "The Setting of Portland Cement," issued by the British Portland Cement Research Association, shows that this body is making investigations which should be of direct value to cement users as well as cement manufacturers. The pamphlet records the results of exposing cement to the action of certain gases existing in the atmosphere, and it is shown that pure dry air has no effect on the setting time; contact with moist air causes cement to become slower setting, and exposure to moist carbon dioxide leads to a marked "quickening up." During storage out of contact with air the setting time of cement remained unchanged.

From these results it is deduced that the effect of exposing cement to the atmosphere depends upon the relative amounts of moisture and carbon dioxide that are absorbed; if moisture absorption preponderates, then the cement will become slower-setting, but if absorption of carbon dioxide is more marked the cement will become quicker setting on exposure. This is an important conclusion, which adds weight to the recommendation of cement manufacturers that cement requires more care in storage by users than it generally receives. It does not seem to have been determined whether upon exposure for the same length of time to the same atmosphere all cements would behave alike or whether some cements are more liable to absorb carbon dioxide and become quicker setting than others.

The effects upon the setting time of cement of various chemical substances were also investigated by the Research Association, and among the results that were to be expected the accelerating effect of sodium carbonate was observed, but the surprising discovery was also made that when crystallised sodium carbonate (containing 63 per cent. of water of crystallisation) is mixed with cement for a few weeks before gauging, the setting time of the latter is not altered. It would appear that under these circumstances the accelerating effect of the carbon dioxide in the sodium carbonate is counteracted by the absorption of moisture by the cement from the crystalline salt.

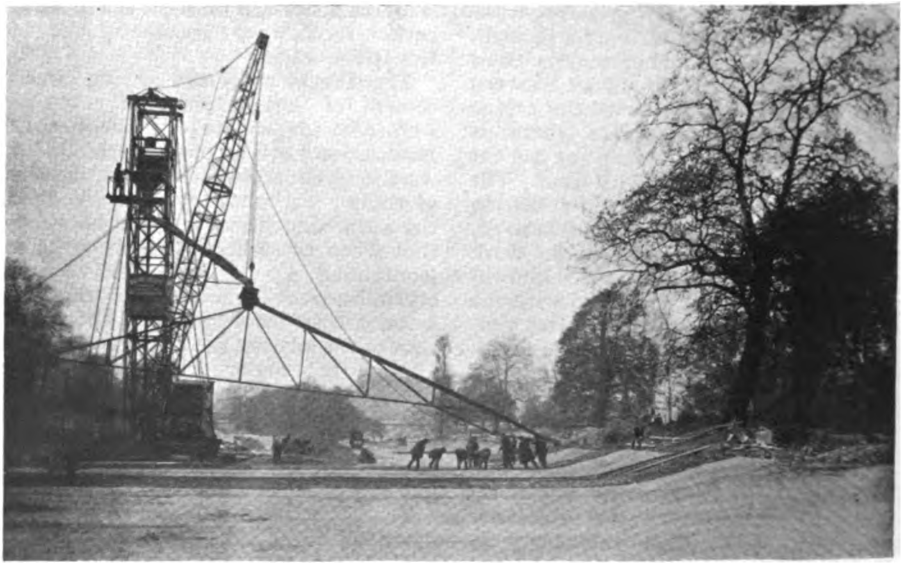
The hypothesis put forward in the pamphlet to explain the changes in the setting time of cement is that a slow setting cement has a protective sheath of hydrated material around its particles resisting or delaying the entrance of water to the active core, and that materials such as carbon dioxide and sodium carbonate, which accelerate the setting time, do so by destroying this protective sheath.

This hypothesis in the light of the Research Association experiments is more tenable than the earlier one that a slow-setting cement depended upon the presence of uncombined lime and that acceleration of setting time was caused by agencies such as carbon dioxide and sodium carbonate which combine with free lime.

The new hypothesis implies, however, that to produce a slow-setting cement from the quick-setting material that always results from rotary kiln production nothing more than absorption of water is needed, but although steam and water hydration are frequently adopted by cement manufacturers it is doubted whether any are able to produce a slow-setting cement without the addition of gypsum, and it is evident that some further explanation is needed to account for the unique action of gypsum upon cement.

CONCRETING THE BED OF ST. JAMES'S PARK LAKE.

The lake in St. James's Park, Westminster, which is now being re-laid with a reinforced concrete bed, has an area of 54,000 sq. yds. After the water was drained off the old bed was broken up by compressed-air plant and consolidated by steam-rollers ready for laying the new bed. The thickness of the new bed is 4 in., and the amount of concrete required is about 6,000 cubic yds., and the amount of steel reinforcement about 200 tons. About 800–900 sq. yds. of concrete 4 in. thick are being laid per day, and it is anticipated that the bed of the lake and all the other work connected with the contract will be finished in July. The concrete is being laid with the aid of two hoisting and distributing towers, one at each end of the Lake bed, mounted on gauge wheels and truck so that when the concrete is laid within the whole radius of the chuting the towers, without dismantling, may be wheeled to the next part of the bed that requires laying. The towers are at present only 60 ft. high, but they are capable of extension to 200 ft. high. The present working radius of the chutes is 100 ft., which, of course, can be increased as the towers are extended. The contractors for the work are Messrs. Wilson, Lovatt & Sons, Ltd., of Wolverhampton. The chuting plant was supplied by the Ransome Machinery Co. (1920), Ltd., of 14–16 Grosvenor-gardens, S.W.1.



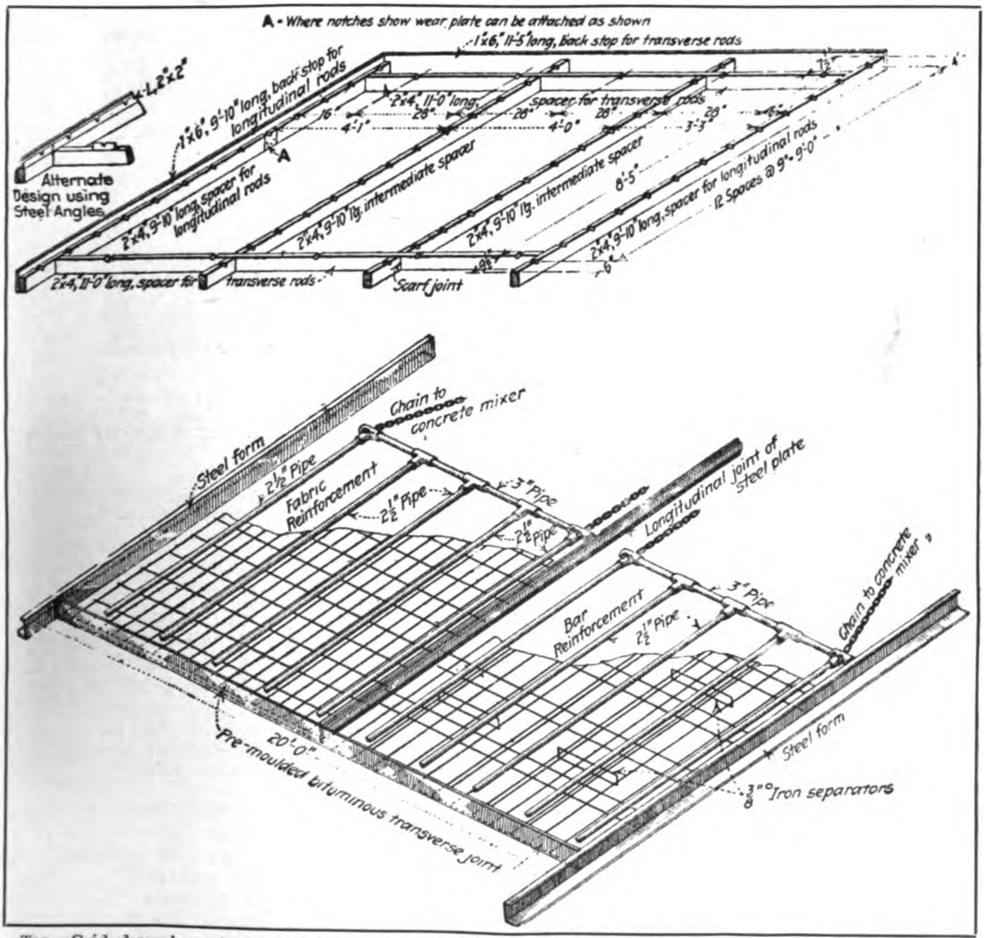
Concreting the Bed of St. James's Park Lake.

ASSEMBLING AND SPACING REINFORCEMENT IN CONCRETE ROADS.

Special methods to ensure the accurate positions of reinforcement in concrete pavement slabs are provided in the standard plans of the New Jersey State Highway Commission. Both pre-woven fabric and assembled-bar reinforcing sheets are employed.

If bar reinforcement is used a wooden frame is set on horses about 2 ft. above the ground so that a workman can step inside between the bars to make the wire ties and for other operations.

Woven fabric is received in flat sheets of the proper size and ready to be put down. When the sheets are assembled they are spread with the proper lap on the sheets previously placed, on the grids of pipe shown in the illustration. These pipe grids hold the sheet off the sub-grade until the concrete is deposited. Then the grids are pulled ahead and another sheet of reinforcement is placed. When there are two layers of reinforcement they are assembled, as indicated, with spreaders to hold them the proper vertical distance apart.—*Engineering News Record.*


A - Where notches show wear plate can be attached as shown
1"x6," 11'-5" long, back stop for transverse rods
L, 2"x2"
1"x6," 9'-10" long, back stop for longitudinal rods
2"x4," 11'-0" long, spacer for transverse rods
16"
28"
28"
28"
28"
6"
4'-1"
4'-0"
3'-3"
1½"
A
Alternate Design using Steel Angles
2"x4," 9'-10" long, spacer for longitudinal rods
2"x4," 9'-10" lg. intermediate spacer
2"x4," 9'-10" lg. intermediate spacer
8'-5"
2"x4," 9'-10" long, spacer for longitudinal rods
12 Spaces @ 9" = 9'-0"
9½"
6"
2"x4," 11'-0" long, spacer for transverse rods
Scarf joint
Chain to concrete mixer
Steel form
2½" Pipe
Fabric Reinforcement
2½" Pipe
3" Pipe
2½" Pipe
Longitudinal joint of steel plate
3" Pipe
Bar Reinforcement
2½" Pipe
Chain to concrete mixer
20'-0"
Pre-moulded bituminous transverse joint
Steel form
⅜" ○ Iron separators

Top.—Grid, drawn by paving machine, for spacing mesh reinforcement. Bottom.—Frame for assembling bar reinforcement.

Placing Reinforcement in Concrete Roads.

TEST ON A REINFORCED CONCRETE. BEAM.

An interesting test was recently carried out at the works of the Siegwart Fireproof Floor Co., Ltd., at Croxley Green, Herts, on one of the well-known "Siegwart" floor beams. The beam tested was 10 in. wide with an overall depth of 4¼ in., with a clear span of 10 ft. between the supports at each end. The weight of the beam was 24 lb. per sq. ft. The reinforcement consisted of two $\frac{5}{16}$-in. rods weighing 11·484 lb. and two $\frac{3}{16}$-in. rods with a weight of 1·974 lb. The materials used for the concrete were 4 parts of ⅜-in. crushed gravel and sand in equal proportions to 1 part of Portland cement. The beam was calculated for a factor of safety of 4, and was tested after maturing for two months. As shown in *Fig.* 1, the beam was loaded with bricks, and the deflection at the centre noted at different loads. The result of the test was as follows:—

Reinforced Concrete Beam loaded at 4·75 cwt. per sq. ft.

Load per sq. ft.	Deflection at Centre.
85 lb.	0 in.
1·14 cwts.	$\frac{1}{16}$,,
1·53 ,,	$\frac{3}{16}$,,
1·9 ,,	$\frac{5}{16}$,,
2·28 ,,	½ ,,
2·66 ,,	⅝ ,,
3·04 ,,	¾ ,,
3·42 ,,	⅞ ,,
4.20 ,,	1 ,,
4·75 ,,	fracture at centre.

Three previous tests were made on similar beams with results approximately the same as that enumerated above.

These beams are calculated for a safe load of ½ cwt. per sq. ft. on a 10 ft. span.

[We shall be pleased to receive for publication particulars of new or improved methods, plant, apparatus, etc., of general interest to our readers.—Ed.]

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Best British Portland Cement	per ton 58*s.* to	63	0
"Ferrocrete" Portland Cement	10*s.* per ton extra		

Boarding for Shuttering—

		Sawn.		*Wrot.*	
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

Sawn Timber for Struts and Supports—

3 in. by 4 in.	from £22 10*s.* per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £23 10*s.* ,, ,,

Mild Steel Rods for Reinforcement—

		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	14	6
9/16 in. to ½ in. Rounds	,,	14	6
⅜ in. Rounds.	,,	15	6
¼ in. Rounds.	,,	17	6

MATERIAL AND LABOUR INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

Portland Cement Concrete well mixed with Clean Water in proportion.

1 : 2 : 4—				s.	d.
Do.	do.	in foundation	per yard cube	45	0
Do.	do.	in columns	,, ,,	50	0
Do.	do.	in beams.	,, ,,	50	0
Do.	do.	in floor slabs 4 in. thick	per yard super	5	3
Do.	do.	in floor slabs 6 in. thick	,, ,,	7	9
Do.	do.	in floor slabs 9 in. thick	,, ,,	11	3
Do.	do.	in walls 6 in. thick	,, ,,	8	0

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

Steel Reinforcement, including Cutting, Bending and putting into Position and securing with Stout Binding Wire—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	27	0
,, 9/16 in. to ½ in.	,,	25	0
,, ⅝ in. to 2⅞ in.	,,	24	0

Shuttering—

		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	54	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	45	0
Do. do. in small quantities	per ft. super	0	9
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	10
Do. do. as last in narrow widths	,, ,,	1	0
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	0
Do. do. as last in narrow widths	,, ,,	1	2
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

MEMORANDA.

Hydro-Aeroplane Hangars at Reval.

THE hangars for hydro-aeroplanes at Reval, described in our May number, were designed and constructed by Messrs. Christiani & Nielsen, of 72–74 Victoria Street, London, S.W.1. The engineer responsible for the design and construction of these hangars is Mr. H. Forchhammer, M.Amer.S.C.E., who is at present the Engineering Manager of the London branch of the firm.

"The Empire Municipal Directory."

WE have received a copy of "The Empire Municipal Directory and Year Book," 1923–24, now in its 41st year of publication. This is a very useful directory and year book for all concerned with municipal work, containing up-to-date lists of all Corporations, County, Urban and Rural District Councils in Great Britain, Ireland and Overseas Dominions, with the names of the respective Town Clerks, Engineers and Surveyors, Gas, Water, Highway Engineers, Health Officers, etc. Special articles are included on Road Construction and Maintenance, Engineering, Building, Public Lighting, Public Cleansing, Water, Sewerage, Public Health, Fire Prevention, Motor Vehicles, etc. Several new features have been added, such as lists of universities and colleges granting Degrees or Diplomas in public health and engineering, and the principal systems of constructing hard tennis courts. The book is easy for reference, and is published by Municipal Engineering, 8, Bream's Buildings, E.C.4, price 10*s*. 6*d*. net.

PROSPECTIVE NEW CONCRETE WORK.

ALTRASNA. — *Bridge.* — The Limavady R.D.C. has decided to replace the bridge at Altrasna, on the Dungiven-Maghera Road, by a reinforced concrete culvert.

BARNSLEY.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the Corporation for sanction to borrow £4,400 and £17,250 for sewerage and sewage disposal works.

BEAULY.—*Pier Works.*—The Inverness-shire C.C. proposes to reconstruct the pier at Beauly, at an estimated cost of £2,000.

BEDHAMPTON. — *Sewage Works.* — The Havant R.D.C. has received sanction from the Ministry of Health to borrow £12,369 for the Bedhampton sewage disposal works.

CHESHIRE.—*Dock.*—Messrs. Lever Bros., Ltd., propose to construct a dock with a water area of 13 acres, together with a lock, entrance jetties, etc., and extensive river walls at Bromborough Port. The cost of the works is estimated at £700,000.

CLACTON-ON-SEA.—*Groynes.*—The U.D.C. has decided to construct a concrete groyne 400 ft. long on the east beach, at an estimated cost of £4,000.

HARTLEPOOL.—*Bridge.*—It is stated that a new bridge over the railway near Hart Station, Hartlepool is to be constructed at an estimated cost of £15,000.

HAVANT.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the U.D.C. for sanction to borrow a further £17,308 for sewerage and sewage disposal works.

HAYLING.—*Sewage Works.*—The Havant R.D.C. has received sanction from the Ministry of Health to borrow £12,369 for sewage disposal works at Hayling Island.

HULL.—*Road.*—The Corporation Works Committee has decided to apply to the Ministry of Transport for a grant towards the cost of a proposed new road to Hessle. The estimated cost is £22,500.

KINGSBRIDGE. — *Culvert.* — The U.D.C. has decided to construct a concrete culvert in Bridge Street, at a cost of about £235.

LANCASTER.—*Esplanade.*—The Borough Surveyor has been asked to prepare plans for the extension of the esplanade at Lune Bank Gardens, along the riverside, to the aqueduct.

LIVERPOOL.—*Road.*—The Mersey Dock Board has expressed its willingness to subscribe £100,000 towards Liverpool's share of the cost of a new road through the Lancashire cotton-spinning districts on to the Yorkshire border.

LLANFAIRFECHAN.—*Groynes.*—The Ministry of Health has held an inquiry into an application by the U.D.C. for sanction to borrow £1,051 for the construction of groynes.

NUNEATON.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the T.C. for sanction to borrow £33,450 for sewerage and sewage disposal works.

ORPINGTON.—*Road.*—The R.D.C. proposes to construct a new road from Hill View Road to Chislehurst Road.

OXFORD.—*Bridge.*—The City Council has decided to apply to the Ministry of Health for sanction to borrow £4,000 towards the cost of reconstructing Seacourt Bridge.

RHYL.—*Defence Works.*—The U.D.C. has decided to apply for sanction to borrow £11,000 for construction and protective works at the Foryd Harbour.

SCARBOROUGH.—*Road.*—The Corporation has received a grant of £3,616 from the Ministry of Transport, being half the estimated cost of the new road from Repney Road to Scalby Road.

SHOREHAM.—*Groynes.*—The Ministry of Health has given its sanction to the Shoreham and Lancing Sea Defence Commissioners to a loan of £4,000 for the construction of four new groynes on the foreshore opposite Shoreham.

SHOREHAM.—*Pier, etc.*—The Works Committee of the Harbour Trustees has approved a report in regard to extensive repair works, and tenders are to be invited for the reconstruction of East Pier in ferro-concrete, estimated cost £3,000; repairs to West and Middle Piers, estimated cost £350; and repairs to lock and canal, estimated cost £500.

STRANRAER.—*Defence Works.*—The T.C. is considering proposals for the construction of a concrete sea wall across the mouth of Clayhole Bay, estimated to cost between £6,000 and £7,000; the construction of a jetty formed of reinforced concrete piles, estimated to cost £8,000, and the construction of a concrete bathing pond, 150 ft. long and 75 ft. broad, estimated to cost £2,000.

WHITCHURCH.—*Sewage Works.*—The Ministry of Health has held an inquiry into an application by the Tavistock R.D.C. for sanction to borrow £5,300 for sewerage and sewage disposal works for the parish of Whitchurch.

WHITEHAVEN.—*Road.*—The Cumberland C.C. has decided to proceed with the construction of the new Whitehaven loop road at a cost of £35,000.

TENDERS ACCEPTED.

BARROW.—*Bridge.*—The Public Parks Committee has accepted the tender of Messrs. Moody & Thompson, at £750, for the construction of ferro-concrete bridges in connection with the park lake extension.

DUBLIN.—*Road.*—The Corporation has accepted the tender of Messrs. Colleen Bros., East Wall, at £1,625, for reinforced concrete carriageways on the Donnelly's Orchard housing site.

HULL.—*Sumps.*—The T.C. has accepted the tender of Mr. J. T. Levitt, at £2,454 4*s.* 8*d.*, for the construction of ferro-concrete sumps at Sculcoates Lane.

ST. NINIANS.—*Tank.*—The Stirlingshire & Falkirk Water Board has accepted the tender of Messrs. J. J. and P. MacLachlan, Larbert, at £3,838 4*s.* 10*d.*, for the concrete, brickwork, earthwork, etc., in connection with the construction of a storage tank of 500,000 gallons capacity.

STEPNEY.—*Foundations.*—The Borough Council has accepted the tender of Messrs. Perry & Co. (Bow), Ltd., 56 Victoria Street, S.W., at £2,663, for the concrete foundations at the Limehouse generating station.

RECENT PATENT APPLICATIONS.

173,738.—T. Tetlow: Manufacture of objects of cement imitating stone and marble.

183,457.—E. V. Preuss: Production of concrete constructional elements.

185,081.—P. A. J. M. Constan and C. Standaert: Construction of jetties, breakwaters, quays, and other structures.

189,745.—C. A. Jaques: Manufacture of Portland cement mortars.

193,269.—A. W. Jeffery: Metal reinforcement for concrete.

193,301.—L., A. I., J. L., and J. L. Hepburn: Mould for sills, lintels, cornices, and concrete blocks.

193,769.—R. Rios and Sociedad Argoufsa del Cemento Armado: Casting concrete floors *in situ*.

194,061.—A. E. Brown: Reinforced concrete walls.

194,738.—T. Rigby: Manufacture of cement.

194,841.—J. Ryan: Machines for manufacture of building slabs or blocks.

194,880.—G. E. Clare: Fireproof floor construction.

194,999.—J. E. C. Van Genns: Production of coloured or other tennis courts.

195,115.—L. W. Goold (Blaw-Knox Co.): Apparatus for moulding concrete structures.

195,324.—A Melville: Spun concrete pipes.

195,326.—J. B. Hilliard: Building and paving blocks and slabs.

195,485.—Pearson Bros., and Campbell Ltd., and R. S. Pearson: Concrete floors, landings, steps and walls.

195,502.—R. H. Annison: Apparatus for forcing cement, grout, or other materials into the interstices of structures or soil.

TRADE NOTICES.

Mr. J. C. Hulbert has resigned from Messrs. Christmas, Hulbert & Walters, Ltd., and is now trading as Cowan Hulbert, with offices at 423 Abbey House, Westminster, London, S.W.1. All matters pertaining to " Insley " engineering service and concrete placing equipment have been transferred to Mr. Hulbert.

An interesting contribution to the road problem has been made by the Walker-Weston Company, Ltd., of No. 7 Wormwood Street, E.C.2, in the form of a brochure fully illustrating and describing some recent all-concrete roads reinforced with the firm's well-known pyramidal interlocked double-layer reinforcement. Some of the arterial roads for the Manchester Corporation, carried out on this system, presented difficult problems in road engineering, as they are laid on a peat formation, and Mr. J. B. L. Meek, M.Inst.C.E., Engineer to the Corporation, contributes an interesting article describing the work of laying the all-concrete roads on such a poor foundation. The brochure also contains illustrated descriptions of concrete roads reinforced on the Walker-Weston system at Middlesbrough, Bridgwater, Dartford, etc., and reinforced concrete road beds at Lambeth, Holborn, Walthamstow, Belfast, Bethnal Green, etc.

The price of the " 7-S Rex " concrete mixer has been reduced to £295 by Messrs. Gaston, Ltd., of Larden Road, Acton Vale, London, W.3. No modification has been made in the specification, and the machines incorporate the " Chabelco " shock-absorbing chain transmission.

Messrs. Gaston, Ltd., have been appointed sole agents for the Barb Engineering Co., for London, Surrey, Kent, Suffolk, Middlesex, Sussex, Essex and Norfolk.

The business formerly carried on by Mr. H. E. Holden and Mr. E. Haworth, trading as the Concrete Unit Co., has been converted into a private limited company, registered as The Concrete Unit Co., Ltd. The firm has erected a new factory in Ingersoll Road, Trafford Park, and the offices now adjoin the new works ; the business will therefore be controlled in future from Trafford Park.

In connection with the construction of the concrete road at Dungeness, near Lydd, Kent, Mr. H. T. Chapman, M.Inst.C.E., County Engineer, is using " Expamet " reinforcement supplied by The Expanded Metal Co., Ltd., of London and West Hartlepool.

" B.R.C." reinforcement was used in the construction of the Uxbridge Road foundation work illustrated on page 362 of our last issue.

At Sandsend, in the North Riding of Yorkshire, a reinforced concrete culvert, 250 ft. long, 11 ft. wide, and 6 ft. high, approximately parabolic in section, was recently constructed. This culvert was designed to carry a newly-tipped earth filling of about 35 ft. deep over the crown. The work was carried out to the design and under the supervision of Mr. G. Davy, A.M.Inst.C.E., the County Engineer, and since the completion of this work a similar culvert has been constructed at Dunsdale. Similar culverts have been constructed at Crimdon Dene, Castle Eden, and Horden Dene for the Durham County Council, and at Ryhope, Jarrow-on-Tyne, and Chester-le-Street. All these culverts were reinforced with " Expamet " supplied by the Expanded Metal Co., Ltd., of London and West Hartlepool.

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CONCRETE
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A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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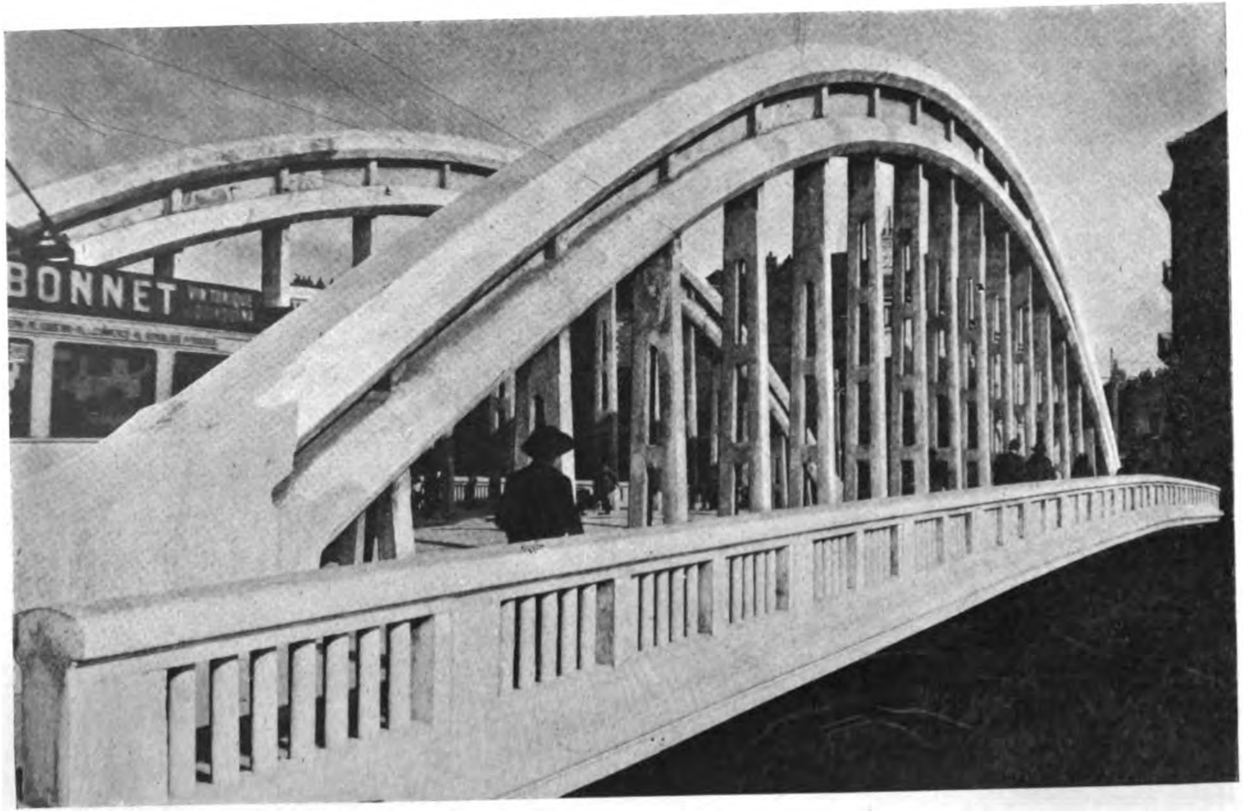

FIG. 5.—REINFORCED CONCRETE BOWSTRING BRIDGE AT NANTES. (*See p.* 439).

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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume XVIII. No. 7. LONDON, JULY, 1923.

EDITORIAL NOTES.

WATERPROOF CONCRETE.

A STRIKING feature of the foreign technical press of recent months has been the number of articles dealing with the degree of impermeability of concrete. The wider uses which concrete is attaining are evidently demanding properties of impermeability, and it is satisfactory that the problem is being attacked abroad on scientific lines. There is abundant evidence that concrete can be, and is, made impermeable, and it is not too much to state that impermeability in fact follows as a matter of course when a properly graded, mixed, and cured concrete is laid.

It has been frequently observed that concrete when first submitted to the action of water under even a slight pressure is permeable and allows water to percolate, but after a brief period not extending beyond a few days the concrete becomes impermeable and no more water passes through it. An extreme case of this change in connection with the resistance of concrete to high water-pressure is cited by Hugentobler (Switzerland), who found that a quarter of a gallon of water passed through a concrete slab under a water-pressure of 73 lb. per sq. in. during the first day of testing, but the flow gradually diminished until at the end of three months the concrete was practically impermeable even at the high pressure which was continually maintained. With the ordinary water-pressures likely to be met with in practice the period during which a new concrete is porous is considerably shorter. It has been suggested that this change may be due in some cases to the pores of the concrete becoming filled with a deposit of carbonate of lime resulting from the interaction of the bicarbonate of lime contained in hard water with the lime set free from the cement during setting. Another explanation is that the lime set free during setting becomes converted into carbonate of lime by the influence of the atmosphere and forms an impermeable skin on the surface of the concrete. Professor White, of the University of Michigan, has, however, made investigations as to the varying porosity of concrete when wet and dry, and he concluded that in a concrete which has been soaked with water the cement is in a colloidal or gelatinous condition, which fills the pores and prevents the passage of water. When the same concrete is dried, the colloidal portion is shrunken and water can pass through the concrete for a short time until the colloids swell again. This conclusion is well supported by the results of White's experiments, and further evidence is again provided that impermeability of concrete will ensue so long as there is sufficient cement present to fill the pores, and in this connection it should be remembered that cement has the advantage of strengthening the concrete.

Maier has investigated the connection between the permeability of a lean concrete (1 to 7 by weight) and the pores as revealed by microphotographs of sections. He finds that imperviousness decreases rapidly as more water is used in mixing the concrete, this being due to the existence of a large number of fine pores in a wet-mixed concrete. On the other hand, if the mixing water is too restricted the sections show that large pores occur which tend to porosity.

The conclusion to be drawn from permeability investigations that have been published is that the rules which must be obeyed to obtain an impermeable concrete are none other than those which govern the production of a satisfactory concrete for any other important purpose, such as road making or reinforced construction. In a few words, the conditions needed are (1) a good cement in the right proportion, (2) a clean and properly graded aggregate, (3) the minimum of water to produce plasticity.

REAL VALUES IN ROAD CONSTRUCTION.

" How long will it last ? " is a natural and reasonable question to ask when a new thing is advocated for use in place of an article to which we are accustomed, and the approximate period over which it will be serviceable is known. In most cases the life of an article is of more importance to the wise purchaser than the first cost, and in the case of things for use which must always be with us it is an indisputable fact that cheapness of first cost is no criterion whatever to the total cost over a period of years, for the renewal and repair of inferior or inadequate articles are likely to be much more expensive than paying a much higher initial price for a sound article with greater wearing qualities.

To this question of " How long will it last ? " when applied to the all-concrete road those interested in road improvement have no reply. They can point to all-concrete roads which have been in service for nearly twenty years without failure ; they can point to the exceptionally heavy traffic many roads of this type are successfully carrying ; they can reiterate the fact that the first trial section of all-concrete road in America eighteen years ago has now grown to some 30,000 miles, and that the number of these roads in Great Britain has increased by 900 per cent. during the past four years—but still there is no answer to the question as to their ultimate life. We think, however, that the experience gained since all-concrete roads were first laid, which is, as already stated, a period of nearly twenty years, should be sufficient to satisfy even the keenest economist with a thirst for exact knowledge as to how long they will last before final failure.

It was something of this desire to know how long an all-concrete road will last which animated the Illinois State authorities when they undertook the most thorough and exhaustive road test which has ever been carried out, known as " The Bates Road Test." The first publication in this country of the results of this test are given in this issue of CONCRETE AND CONSTRUCTIONAL ENGINEERING, and we have tabulated the data for easy reference. The test, as will be seen, was very comprehensive, covering some sixty-three different types and variants of usual forms of road construction. The vehicles used were of just the type which is causing so much concern to those responsible for building roads to withstand present-day traffic—namely, heavy and fast-travelling mechanically-propelled lorries. The weight of the lorries, unloaded, was 4½ tons, and they were loaded with progressively increasing weights until the gross load reached a total of 12⅔ tons. These lorries made a total of 23,000 round trips, and, as will be seen, of the 13 roads which at the end of the test showed no decisive failure 10 were unsurfaced concrete, 1 brick on a 5½-in. bed of concrete, and 2 were concrete foundations, 7 in. and 8 in. thick with 2-in. and 1-in. asphalt surfaces respectively. That these three surfaced sections which survived to the "No Decisive Failure" stage all had a concrete base thicker than some of the unsurfaced concrete roads in the same class clearly indicates that the additional expense of the topping is not warranted by any extra life.

Apart from the outstanding superiority of the all-concrete road over every one of the other sixty-three types of road construction which formed the subject of the test, one or two points in connection with the sections which survived the test are worthy of note: The proportioning of the concrete in all cases was 3½ parts of coarse material to 2 parts of sand to 1 part of Portland cement. Six of the sections were plain and four strengthened with circumferential reinforcement. Two of the reinforced sections were 5 in. thick and two 6 in. thick; of the plain concrete sections, three were 7 in. thick, two 8 in. thick, and one 9 in. thick. No particulars are given as to the sub-grade, but presumably the foundation was a good one and thoroughly consolidated. The trial sections were all 18 ft. in width, with lengths varying from 100 ft. to 250 ft. Of the ten all-con-

[*Mr. Glenn Allan, Architect.*

DETAIL OF CONCRETE WAREHOUSE IN CALIFORNIA: SHOWING CAST-CONCRETE DECORATIONS COLOURED WITH ENAMEL.

crete sections which survived the test, eight had longitudinal joints, strengthened with steel rods connecting the two sides, for the full length. Of these eight sections, four also had transverse joints every 25 ft. Two of the ten sections had neither longitudinal nor transverse joints ; both these sections were unreinforced.

So far as the test has gone it does not seem that any clear decisions can be formed as to the relative advantages of plain or reinforced concrete, of the advisability of providing longitudinal or transverse joints, or of the shock-resisting qualities of different thicknesses, for the sections still in good condition include practically all the different types. The test has served a valuable purpose in showing which type of road is most suited for carrying modern traffic, and in congratulating the Illinois State authorities on their public-spirited and painstaking enterprise we also express the hope that the test will be continued until the sections which have so far survived are worn to destruction. If this can be done some valuable information would be available on several points on which different opinions are now held—and, incidentally, we should have an answer to the question, " How long will it last ? "

THE BUILDING INDUSTRY.

ALTHOUGH the building industry can fairly expect an increased activity as a result of the recent wages reductions, it is probable that the most benefit will accrue from that part of the agreement which definitely fixes the wages until January, 1924, and provides that from that date they shall continue to be based on the Board of Trade cost-of-living index figure. High cost has undoubtedly deterred many from engaging in building, and this is particularly noticeable in the case of the smaller class of dwelling-house, but at the same time a very large number of building schemes have been held up because of the uncertainty as to future costs. The experience of those who purchased commodities of a permanent nature at top prices three years ago, and have since seen values depreciate by anything up to 50 per cent., is still borne in mind by those who are prospective purchasers at the present time, and no one can be blamed for hesitating to buy on a falling market. It can now be claimed, however, that the cost of building will show practically no reduction until 1924 : wages, which account for such a large part of the total cost, are definitely fixed ; and it is not likely that the price of materials will in the near future be reduced sufficiently to make any appreciable difference, for while it is true that some goods are becoming slightly cheaper others show a tendency to harden in price. It seems, therefore, that present day prices are likely to rule for some time to come, and it is to be hoped that now a measure of stability has been reached those who are holding up building schemes for lower prices will realize that no advantage is to be gained by further delay.

REINFORCED CONCRETE BOWSTRING BRIDGE AT NANTES.

By W. L. SCOTT, A.M.Inst.C.E.

THE bowstring type of bridge is one containing many features not possessed by other classes. It also proves to be in many cases more economical in first cost owing to the elimination of complicated and heavy abutment supports.

This type of bridge consists essentially of two or more elastic arch ribs similar in design and in manner of working to those employed for ordinary elastic arch bridges having similar conditions of end restraint, the fundamental difference being that the thrusts that would ordinarily be imposed at the abutments are taken by " strings " or ties connecting the ends of the arch ribs. These ties are suspended from the arc by vertical hangers or suspenders at suitable intervals.

It is usual in practice to form the deck platform at the level of the ties, and when bowstring girders are introduced bridges capable of spanning considerable distances without any projections below the deck construction can be employed.

One of the most recent and striking examples of this class of bridge is that illustrated herewith, which has been constructed to a Considère design. It has a clear span of about 180 ft., and an elevation of the bridge is shown in *Fig.* 1.

The elastic arcs or rib members are usually constructed of parabolic form having a rise approximately equal to one-sixth of the span. The suitability of this type of bridge possessing two ribs is limited to reasonably narrow construction owing to the necessity for carrying the roadway by a series of transverse beams, which, when the width of the bridge becomes great, project below the underside of the roadway or rail tracks and may possibly interfere with the passage of traffic under the bridge.

For bridges of ordinary widths required in practice, however, the bowstring girder bridge is quite suitable, and where a fairly wide road is required the bowstrings may be placed on the inside edge of the pavements and the footpaths themselves cantilevered out on each side. This latter arrangement was adopted for the bridge at Nantes, and *Fig.* 2 shows this and makes clear the accommodation available for pedestrian traffic.

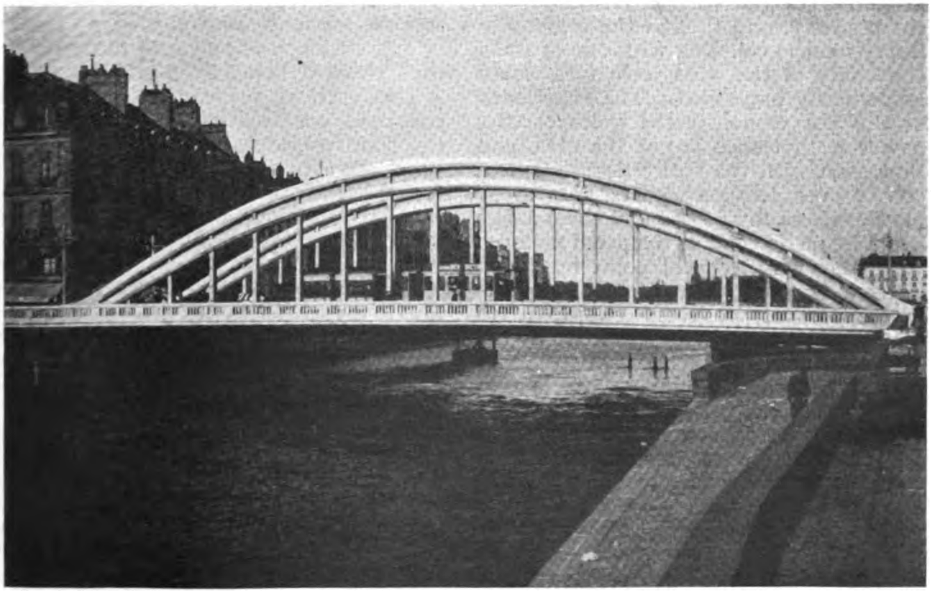


FIG. 1.—ELEVATION.

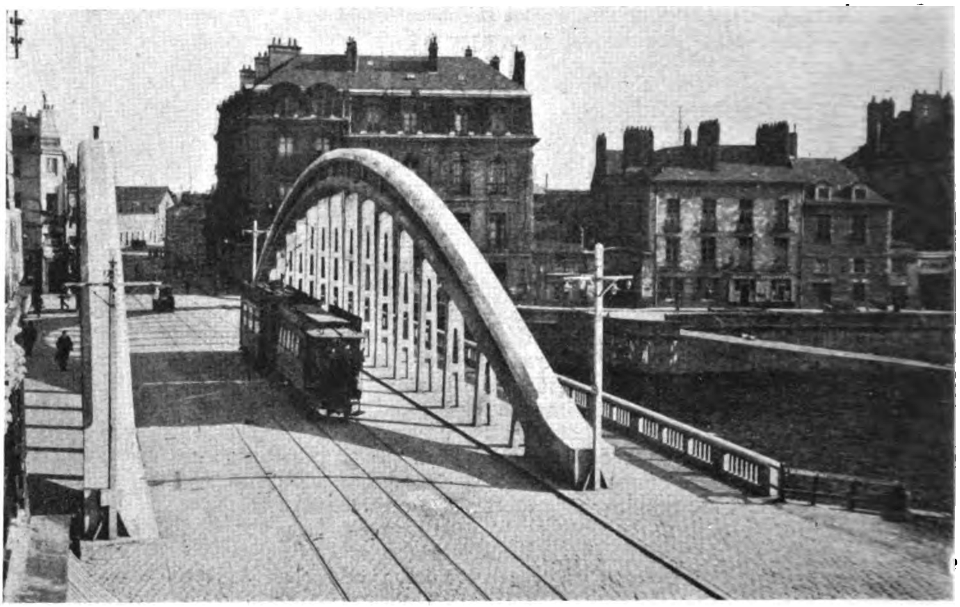


FIG. 2.—VIEW LOOKING ALONG BRIDGE.

When the particular arrangement above described, in which the bowstring girders are placed adjacent to the roadway, was first introduced, it was thought that some danger might result from the proximity of the vertical suspenders to the roadway, and that in the event of a heavy vehicle colliding with one of them the safety of the whole structure would be seriously imperilled. The experience of the war demonstrated quite clearly, however, that such is not the case, and that even with the entire destruction of one or even two suspenders, the bridge platform was unaffected and the damage could be made good in the usual manner. A further controversial point regarding the employment of reinforced concrete for important members subjected to pure tension is indisputably settled by the complete success of this class of structure.

Many engineers and architects not fully conversant with the characteristics of properly-made concrete employed in the construction of modern reinforced concrete work have from time to time expressed doubts as to its ability to withstand pure tension. Such doubts, however, are shown to be without justification having regard to the above structure, and to the many examples of bridges and arch roofs of large span that have been erected during the past few years, in which important structural members are subjected to purely tensile stresses. These structures, without exception, have given entirely satisfactory service without any untoward results or any evidence whatever of the concrete thus stressed being unsuitable.

The vertical suspenders from which the road construction is suspended in the Nantes bridge, and all others of its class, are subjected to pure tension, and under certain wind conditions they are subjected to additional tension on one face due to the bending effects produced.

The question of wind-pressure is important in these bridges, and the lateral effects of the maximum assumed wind-pressure upon the exposed elevations require to be carefully taken into account. The total horizontal reaction resultant from wind-pressure upon the bridge girders has obviously to be transmitted through the deck construction to the abutment supports. For this reason the deck has to act as a horizontal girder, and requires to be designed as such so far as these lateral forces are concerned. The manner in which the deck platform is loaded depends to some extent upon the form of attachment to it of the curved ribs.

These ribs may be braced lattice-fashion so as to form a curved lateral girder and its end reaction carried down to the deck construction by transverse framing, which is arranged near each end of the bridge at sufficient distance from the supports to permit of the required headroom. This arrangement, however, is in ordinary circumstances not adopted for several economical and practical reasons, and as far as possible, this class of bridge is designed so that each girder carries its own share of the lateral wind-pressure coming upon it and the ribs are not connected in any way. As will be seen from *Fig.* 2 no connection is made between the two curved ribs to increase the resistance of the windward members against lateral pressure. Each of these members is made stable by the form and design of the vertical suspenders themselves.

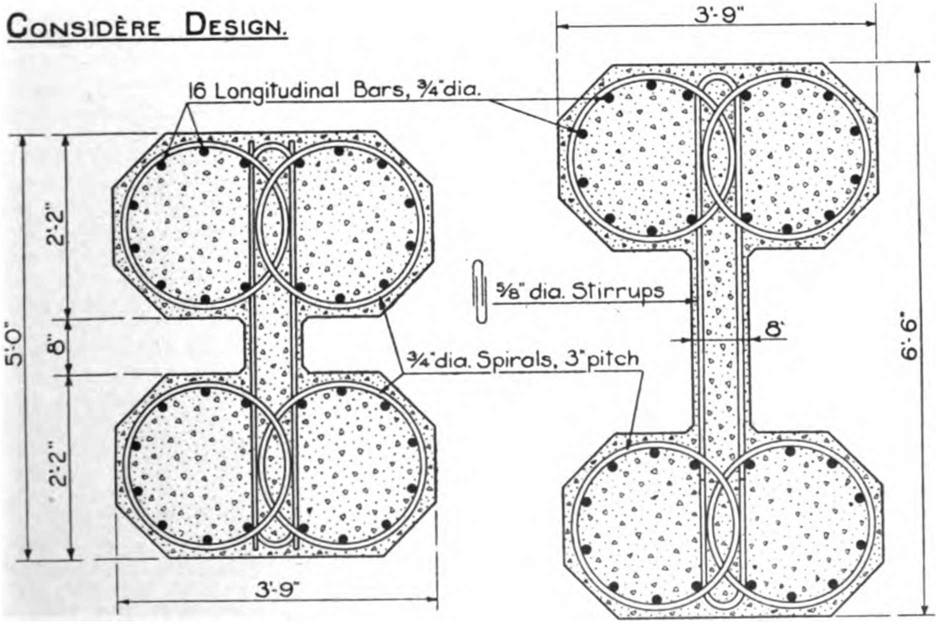
The bending due to the superimposed loading in the bowstring girders is partially taken by the curved rib and partly by the horizontal longitudinal member forming the tie immediately below them at the deck level.

Where the inertia of this latter member is very small compared with that of the rib it is usual to omit its effect; but in some cases where the outside girders are in line with the parapets the horizontal member may be of considerable stiffness, and it is then necessary to take into account the vertical bending of both the curved rib and the above horizontal parapet girders. When this is done, the assumption is made, in calculating the bending of the entire girder, that the length of the vertical ties or suspenders remains unaltered.

In the case of this bridge, the depth of the ribs is increased from 5 ft. at the springings to 6 ft. 6 in. at the crown, and they are provided with a constant maximum width of 3 ft. 9 in.

Fig. 3 shows cross-sections of the ribs at these positions, and the arrangement of the reinforcement in the curved rib is also clearly indicated.

This reinforcement is composed of two pairs of interlocked spirals running throughout the length of these curved members, enclosing longitudinal rods in the manner usually adopted by Messrs. Considère Constructions, Ltd., in their



Section of Rib at Springing.

Section of Rib at Crown.

Fig. 3.

designs. These members may be regarded as of **H**-girder section in which the two pairs of spirals form the top and bottom flanges connected by a thin web 8 in. thick, stiffened at intervals in the usual manner.

Probably the most important point in designing and constructing a bridge of this class is the connection between the horizontal tie and the curved rib at the abutments. It will be realised that a very considerable thrust is given by the rib members at the springings, and the connections to these points have to be studied very carefully and the design so arranged that the tie is capable of absorbing the whole of the thrust in the relatively short distance available at these points of contact.

The bridge at Nantes, having a span, as already mentioned, of about 180 ft., produces a most serious thrust in the arch ribs, and the connections at these points consequently become of vital importance and require very considerable care and experience in the design and arrangement of the steel reinforcement situated at and immediately around them.

The total quantity of steel reinforcement provided in the bridge amounts to 212 tons, which is made up in the following manner :—

	Tons
Bowstring girders	139
Deck construction	67
Abutment supports (not including reinforced concrete piling)	6
TOTAL	212

The greater percentage of the steel to the bowstring girders is placed in the horizontal ties.

The variations in the length of the bridge due to rise and fall of temperature have been provided for, and at one end of the bridge provision is made for angular movement ; at the other end cast-steel roller-bearings are introduced, which permit lateral as well as angular movement to take place, thereby eliminating all possibility of unprovided-for stresses being set up from expansion or contraction due to temperature variations.

The bridge has an over-all width of 65 ft. 7 in., with a roadway of 32 ft. 9 in. carrying a double tramway track.

The lateral spacing of the bowstring girders is 39 ft. 5 in. centre to centre.

The curved ribs are constructed with a rise of 31 ft. from the centre of the main tie beams to the centre of the ribs. It will be seen that this dimension gives a rise span ratio of about one-sixth, this being, as already mentioned, the figure usually adopted for this class of structure.

The whole of the deck construction, including the ties, is formed with a longitudinal camber of approximately 1 in 50, and the transverse deck beams and vertical suspenders are spaced at 9 ft. 9 in. centres throughout the span. The latter members are 9 in. thick on elevation, tapering in width from 2 ft. 8 in. at the deck level to 1 ft. 9 in. at their upper ends.

As will be seen from the illustrations, these members are perforated, as are also the cantilevered portions of the cross-beams at deck level. The material omitted is not structurally necessary, and although inconsiderable, it assists in reducing the dead-weight of the structure, the latter providing, of course, by far the most serious loading to be carried.

The section of the main curved ribs, as described, is also designed with a view to giving the greatest possible strength with the absolute minimum of material necessary to ensure proper safety.

It has already been stated that the cross-beams are spaced 9 ft. 9 in. apart, and these form the principal supporting members of the deck slab carrying the roadway. This portion of the deck slab is 6¼ in. thick at each curb, increasing to 7 in. at the crown of the roadway, giving an average of 6⅝ in.

The roadway carries, in addition to the tramway tracks, the various assortment of vehicular traffic usually found in busy towns, including train loads comprising vehicles giving wheel loads of anything up to seven tons. This latter figure does not include any allowance for impact, which is properly provided for by making a percentage reduction in the working stresses for the portions of structure immediately interested and affected by load vibrations. This fact is interesting in view of the popular impression of the limited capability of reinforced concrete slabs to sustain heavy moving-point loads.

It is important to note, however, that although the thickness of the slab is comparatively small, the percentage of steel reinforcement is correspondingly high, a large proportion of this material being placed in the longitudinal direction to take up the inevitable bending moments developed in this direction.

The amount of surface area of slab interested in carrying a concentrated load having a relatively small contact surface is much larger in a reinforced concrete slab than is generally realised. Recent investigations of eminent engineers of the Ponts et Chaussees of France have demonstrated this to be the case, and the previous regulations obtaining in France upon the subject have been passed over in favour of the more correct methods now obtaining. Although this is the case, the older methods now discarded in France are still considered in this country to be somewhat daring.

Conservative methods, where they tend to increase the dead weight of the structure, are not necessarily upon the side of safety, and in the absence of any contrary evidence of any kind forthcoming from reliable sources in this country the methods so successfully employed in France may be justifiably adopted here as a safe standard.

The increasing number of examples of large and important bridges in that country designed by French engineers place them in a progressively superior position to engineers in this country.

It may be of interest to state that the original design for this bridge, prepared before the war, was for an arch type of structure. Messrs. Considère, who prepared this earlier design, were afterwards asked to submit an alternative scheme in which the cost would not exceed the amount originally contemplated, taking into account the depreciation in value of the franc.

The bowstring girder bridge described above was therefore put forward and adopted. The cost of constructing the latter type of bridge was as anticipated and a great saving in cost was thereby realised, although the latter structure is probably not quite so handsome or impressive as the structure originally contemplated.

[A perspective view of the bridge is given in our Frontispiece.]

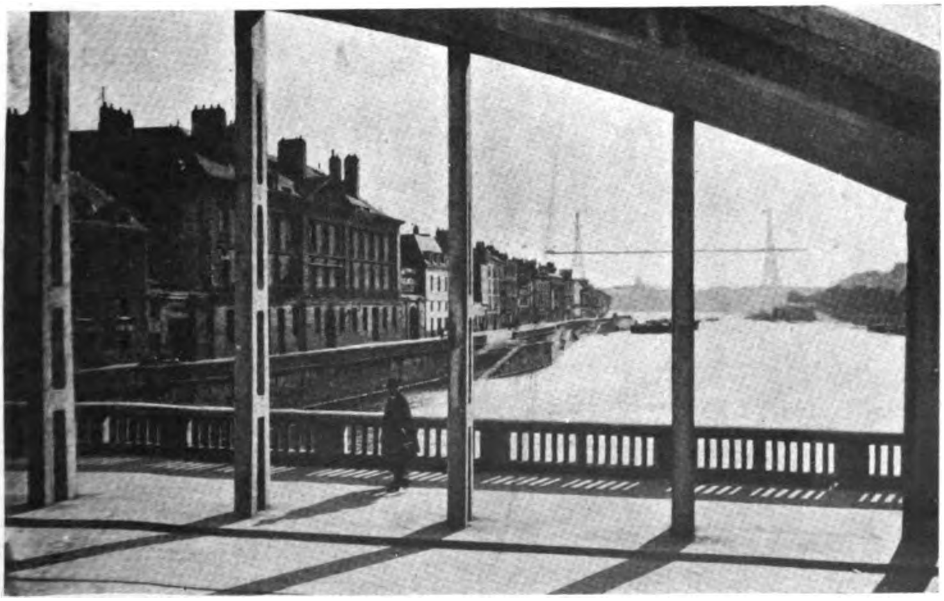
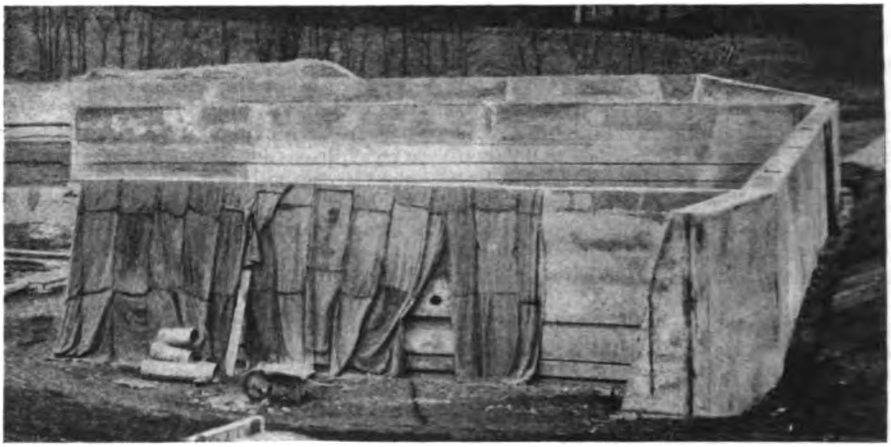


FIG. 4.—VIEW THROUGH SUSPENDERS.



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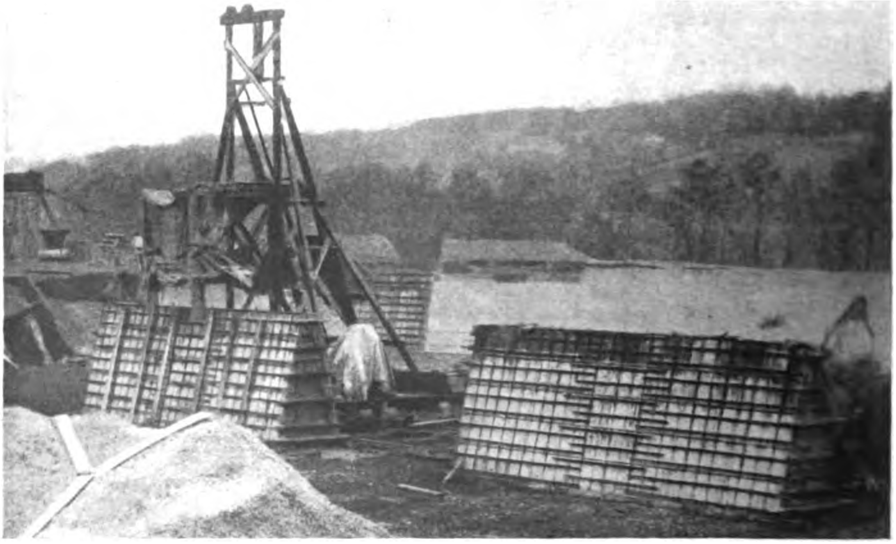


Filters during Construction: Showing Blaw Light Wall Forms.

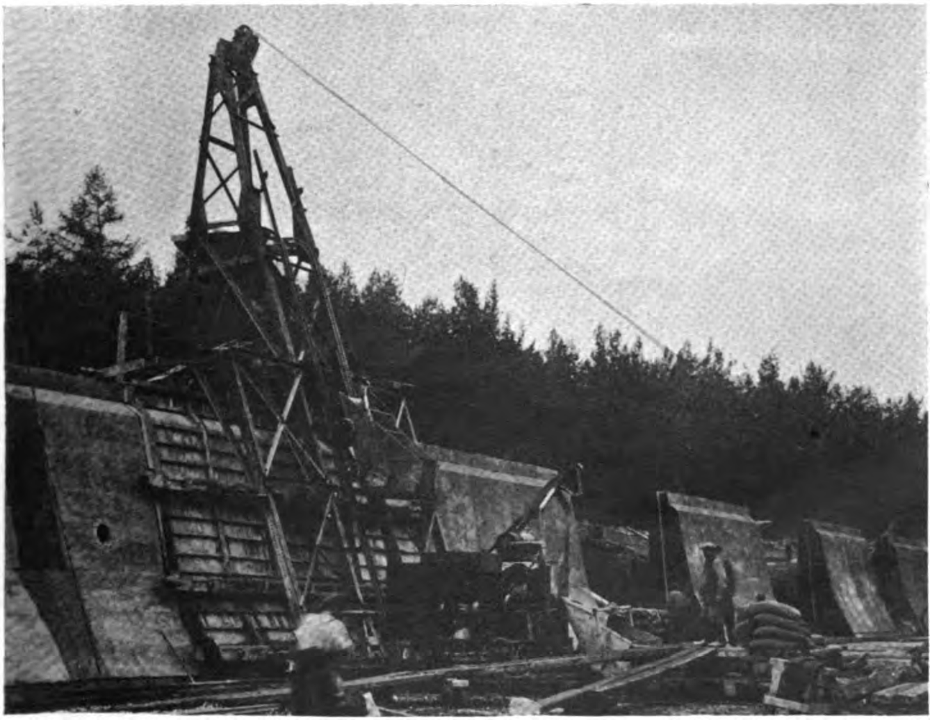
NEW WATER SUPPLY SCHEME AT ABERDEEN. (*See p.* 445.)

NEW WATER WORKS FOR THE CITY OF ABERDEEN.

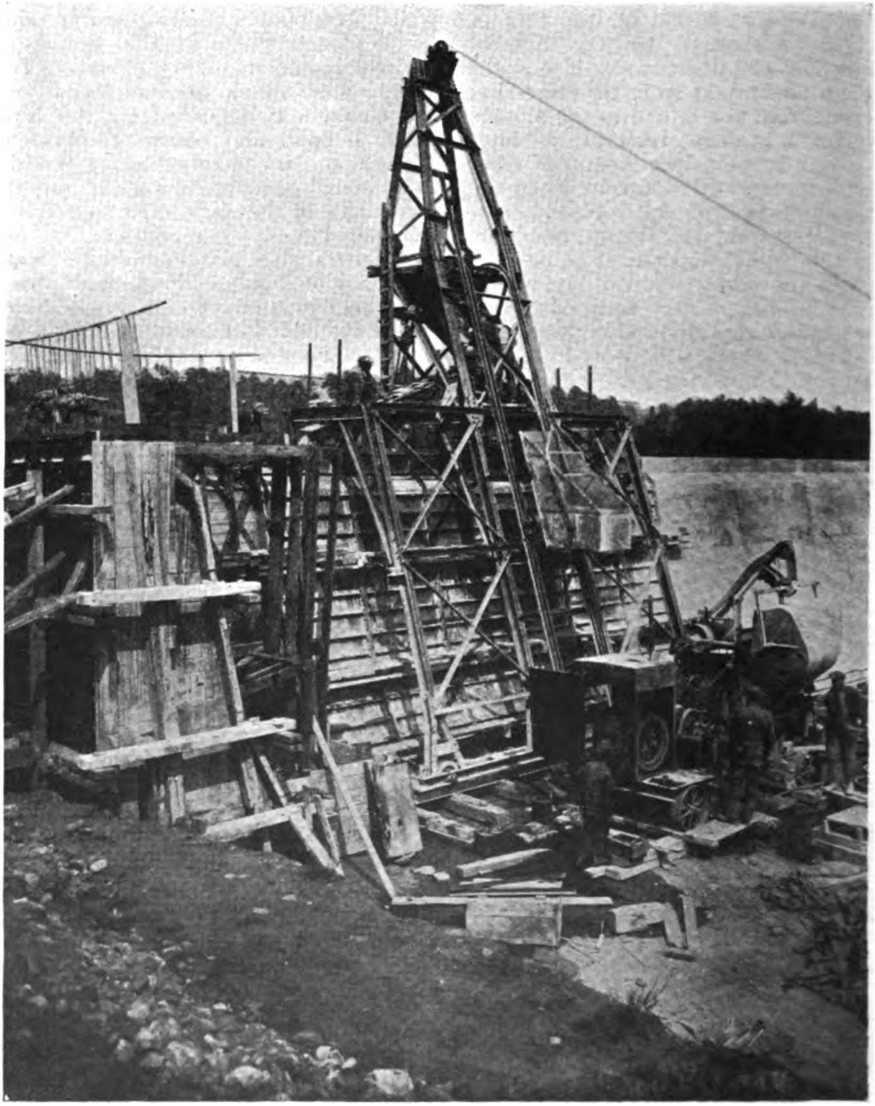
THE new water scheme for the City of Aberdeen, authorised by the Act of 1916, is now well under way. The works include a new intake from the river Dee, a conduit from the intake to the storage reservoirs; storage reservoirs; filter beds; a new aqueduct from the filter beds to the service reservoirs near the City; additional pumping plant; new trunk mains from the service reservoirs to the City, etc. Concrete is being used largely in the construction of the works, and we illustrate herewith the new storage reservoir and filter beds at Invercannie, which will, it is expected, be completed this year.

The storage reservoir is 660 ft. long by 390 ft. broad, has an average depth of water of 15½ ft. and a capacity of 24 million gallons. The walls and floor are constructed of mass concrete, the walls being made in alternate sections, 30 ft. long. Each of the sections is entirely completed in a day. Travelling steel forms are employed, and the concrete is deposited by the usual hoist, hopper, and chute arrangement. Along with the concrete a facing of 1½ in. of cement mortar is worked on the water face of the walls. The sections of the walls have " U " recesses at the ends and vertical steel plates across the joints. The ends of sections are painted with a thick coat of a mixture of tar and pitch before the intermediate sections are filled in. The floor is covered with asphalte worked into a groove near the toe of the walls.

The reservoir is divided longitudinally into four bays by means of reinforced concrete baffle walls in order to ensure thorough circulation throughout the water

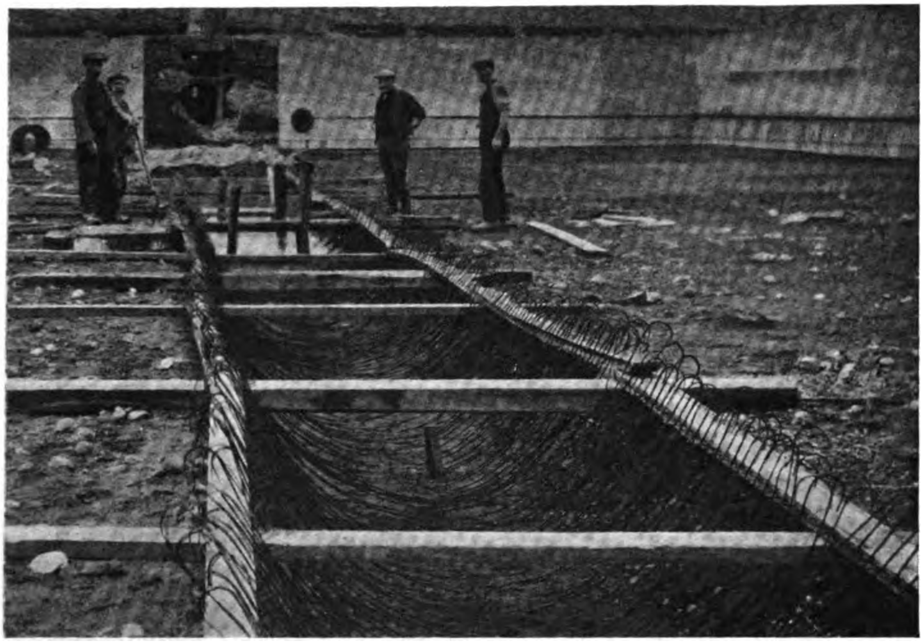


NEW WATER SUPPLY SCHEME AT ABERDEEN: BLAW FORMS, TRAVELLING TOWER, MIXER AND HOIST. (SHOWING CONSTRUCTION OF RESERVOIR WALLS IN ALTERNATE SECTIONS.)



[*Mr. George Mitchell, M.Inst.C.E., City Water Engineer.*

New Water Supply Scheme at Aberdeen: Reservoir Wall during Construction (View from Inside).



New Water Supply Scheme at Aberdeen: Main Collecting Drain during Construction.

in the reservoir. The water enters the reservoir from a new concrete conduit, 6 ft. high by 3 ft. wide, half a mile long. The outlet and by-pass pipes are of reinforced concrete, 4 ft. in diameter, made on the site. The usual draw-off well is provided. The overflow and wash-out pipes are of reinforced concrete and were made on the site.

The filter beds are of the usual slow sand type. There are three beds with a total sand area of 6,800 sq. yds. The walls are constructed of mass concrete, in 25 ft. lengths, with asphalte keys between the lengths. The floors are of reinforced concrete. The total filtering area available is now 16,500 sq. yds.

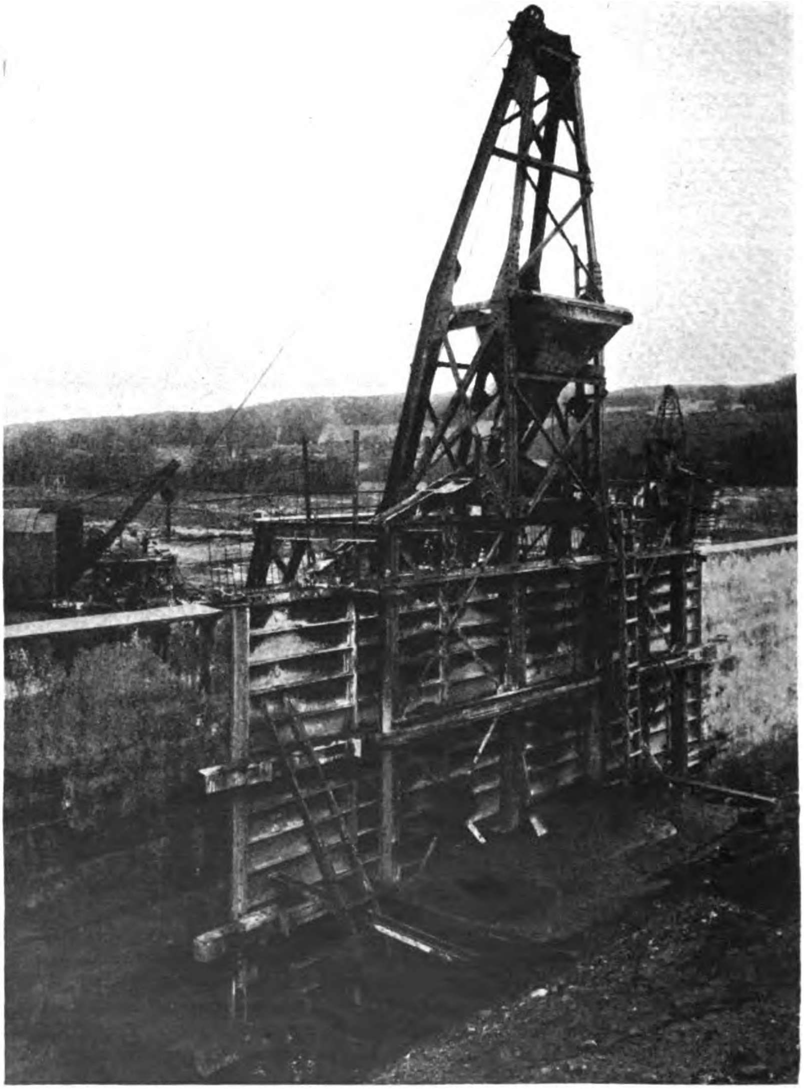
The waste water from the works at Invercannie is passed through concrete settling tanks before being discharged into the River Dee.

The new aqueduct will be 17½ miles in length, and will have a capacity of 14 million gallons per day. A length of 5 miles is now under construction, partly of concrete, semi-elliptical or circular in cross-section. Where under pressure, steel pipes lined with cement mortar are employed.

The new waterworks are being carried out from the designs and under the supervision of the City Water Engineer, Mr. George Mitchell, M.Inst.C.E., M.I.Mech.E., F.G.S. The estimated cost of the scheme is £1,125,000. The contractor for the storage reservoir, filter beds and part of the new aqueduct is Mr. William Tawse of Aberdeen.

The Steel Forms used on the filter walls are Blaw standard light wall forms, which are adaptable to both light and heavy work, the bracing being taken care of by angle liners attached as required.

The reservoir walls were constructed with Blaw travelling wall forms designed and supplied by the Blaw Knox Co., of Pittsburgh, Pa., whose sole concessionaires for Great Britain are Messrs. Christmas & Walters, Ltd., Caxton House, Westminster, London, S.W.1.



[Mr. George Mitchell, M.Inst.C.E., City Water Engineer.

NEW WATER SUPPLY SCHEME AT ABERDEEN: OUTSIDE OF RESERVOIR WALL DURING CONSTRUCTION.

(See page 445.)

SILICATE OF SODA AND CONCRETE.

By L. A. MUNRO, B.A., F.I.C.

In an article published in this Journal in April, 1921, attention was drawn to silicate of soda as a concrete " improver." It was pointed out that silicate of soda is fundamentally sound as an improver of concrete because it provides active silica which can combine with lime set free during the setting process of cement and thus form silicate of lime, which is one of the compounds contributing to the strength of cement. It was stated that silicate of soda is otherwise known as " water glass," and is retailed as a viscous liquid for egg preservation.

Recently in this Journal (April, 1923) it was pointed out that there are several forms of silicate of soda on the market, the most useful for treatment of concrete being known as " 3·3 ratio," meaning that the solution contains 3·3 molecules of silica for one molecule of soda. Most authors, however, refer to " water glass " as a " viscous liquid " (See *Concrete and Constructional Engineering*, 1922, p. 685).

One of the objects of this note is to point out that the viscous liquid " water glass " of commerce such as is generally used for egg preserving and the form of silicate of soda which is the correct one for concrete treatment differ very materially from each other, both in chemical composition and in properties. What is really required for concrete treatment is the silica and not the soda.

All the forms of silicate of soda in commercial use contain more silica than that indicated by the formula of the meta-silicate Na_2SiO_3, the ratio between soda (Na_2O) and silica (SiO_2) varying between 1 to 4 and 2 to 3.

The commonest forms on the market consist of solutions in which the ratio of soda to silica is either 1 to 2 or 1 to 3·3. It is the solution with three or more parts of silica to one of soda which should be used for the treatment of concrete. Such " neutral " or " high ratio " silicates are put on the market in the form of clear solutions which are not so viscous as the low ratio solution, generally known as " water glass," and are much easier to handle. It is now possible to obtain a " neutral " silicate of soda in the form of a white powder soluble in hot water, and this is a very convenient product for the small user.

The types of silicate of soda referred to in this article are solutions which can be purchased wholesale and which contain approximately 30 per cent. of silica (SiO_2) and 9 per cent. of soda (Na_2O), and a powder soluble in hot water and containing about 60 per cent. of silica and 20 per cent of soda.

GENERAL ACTION OF SILICATE OF SODA ON CONCRETE.

When a *dilute* solution of " neutral " silicate of soda is applied to concrete, some absorption occurs according to the degree of porosity of the concrete. The surface of the concrete is thereby hardened and the pores of the concrete are to some extent filled. The silicate of soda slowly undergoes chemical changes and insoluble products are formed which act as a permanent filling near the surface, and form a dense layer, perhaps ⅛ to ¼ of an inch thick, so that the concrete is more resistant to abrasion and more impervious to water and other liquids.

THE HARDENING OF CONCRETE FLOORS TO PREVENT ABRASION AND DUSTING.*

The aim of the treatment in this case is to get the silicate absorbed in the concrete so that it can react, fill up the pores, and support the surface against abrasion.

The U.S. Bureau of Standards † recommend that the silicate solution should be diluted by adding four volumes of water to one of silicate, and that three coats of this dilute solution should be applied.

The Bureau of Standards report states that the treatment with an 8 per cent. solution of commercial sodium silicate produced a panel which after twenty-six months showed no signs of wear, and draws attention to the inexpensive nature and simplicity of the process.

It is essential that the floor should be clean and free of grease before treatment, and in order to obtain satisfactory penetration it must be dry.

The floor must again be allowed to dry

[*Continued on p.* 452.]

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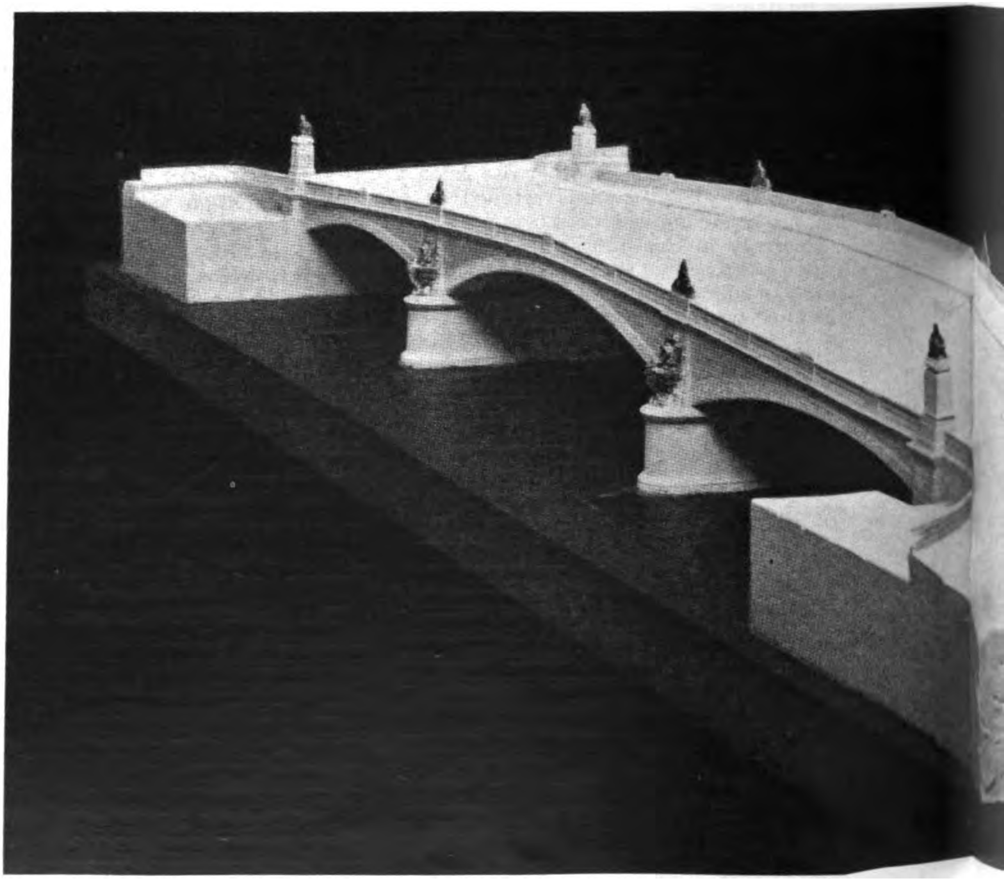
* " Report on Service Tests on Concrete Floor Treatments." Bureau of Standards, Washington, D.C. Oct. 1920.

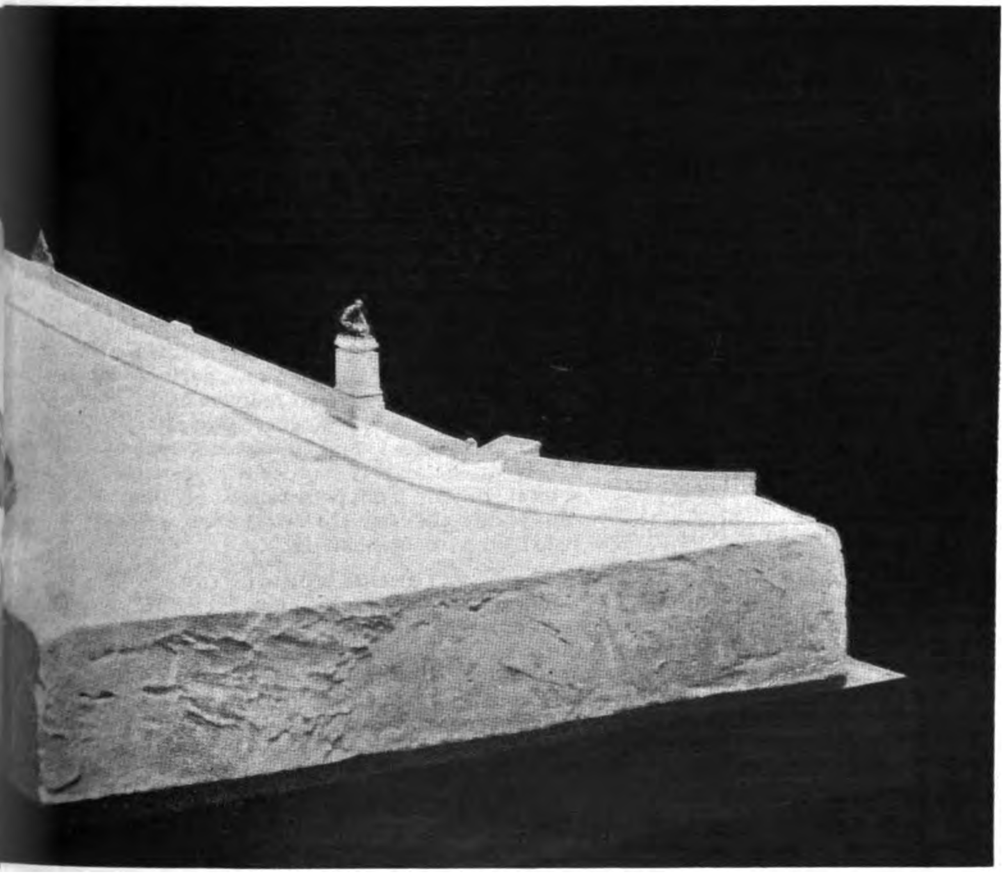
† Moyer—*Concrete-Cement Age*, IV, (1914).

OSWALD STREET BRIDGE, GLASGO



The main dimensions of the design, which is by Messrs. Considère Constructions, Ltd., are as follows: Side spans, 122 ft.; centre span, 165 ft.; rise of centre span from the springings to underside of arch, 14 ft.; width between parapets, 80 ft. In order to make provision for the expansion and contraction under variations of temperature it is proposed to carry the whole of the spans on steel roller bearings of a type similar to those often adopted in steel bridges. The river piers will be founded on three circular caissons, 20 ft. in diameter, made of reinforced concrete,

LASGOW: SELECTED DESIGN.



and provided with a steel cutting edge. The shore abutments will be carried on three caissons, 16 ft. in diameter. At the request of the City Engineer, Messrs. Considère Constructions, Ltd., have allowed for the whole of the bridge being faced with granite, and the parapets will also be constructed of this material. When completed the bridge will be the most ambitious project yet carried out in this country in reinforced concrete.

[*Concluded from p.* 449.]

before the second and third coats are applied.

One gallon of silicate diluted with four gallons of water should cover 1,000 sq. ft. of floor surface for one coat, and the cost of such treatment should be less than 9*d.* per 1,000 sq. ft. for each coat.

Concrete floors treated in this way have a bright appearance, are very hard and are not slippery.

For quick work one coat only with a stronger silicate solution gives satisfactory results if repeated at regular intervals.

CONCRETE ROADS.

Silicate treatment is eminently suitable for concrete roads and will probably be more generally used when the results of some tests now being made are known. Silicate treatment will always improve the texture of the road surface without any fear of making the road slippery.

It is cheap and easily carried out. The diluted solution can be applied by a water-cart or through the rose of a watering-can, and should be brushed over the surface with a soft broom.

About a fortnight after laying is said to be the best time for treatment, and the road must, of course, be as dry as possible before the silicate coats are applied.

Silicate treatment immediately after the initial set has been suggested in order to prevent evaporation from the surface and retain the moisture which is necessary during the initial stages of the setting process, but no information is available as to whether this treatment is always successful. This process may be useful in cases where water is scarce.

WATERPROOFING CONCRETE.

A great deal has been written both for and against integral and surface waterproofing treatments, and, although good materials and workmanship are the first essentials in making waterproof concrete, silicate treatment of the surface will tend to counteract imperfections.

For this purpose the first coat of silicate should be very dilute, about five volumes of water to one of neutral silicate of soda solution being used. This should be followed up with four other coats of a rather less dilute solution made by adding three volumes of water to one of silicate.

Building blocks may be hardened and rendered denser by pickling in dilute silicate solution.

CONCRETE OIL TANKS.

In a paper read before the Convention of the American Concrete Institute it was recommended that all reinforced concrete oil tanks should be given a treatment with silicate when first built, as the oil interferes with the setting of the cement.

One treatment recommended consisted in giving a first coat with a solution made by adding three volumes of water to one of silicate, and this was followed by a coat made up by adding only two volumes of water to one of silicate. Two final coats are then made with a stronger solution still applied by mixing equal volumes of silicate and water.

Moyer,* referring to the treatment of concrete vats, says :—

" After the forms are removed projections are ground off with the carborundum stone. The surface is kept damp for fourteen days, washed and dried, and then washed with a solution of one part of 40° Be. silicate of soda to four or six of water. The surface is washed again after a period of from four to twenty-four hours, dried, and the process repeated several times. The penetration of the silicate into the concrete varies from one-eighth inch to one-half inch."

PATCHING WORK.†

Silicate of soda should not be added to cement or concrete when gauging if the tensile strength of the finished product is of major importance.

A small amount of silicate renders cement very quick setting, and for patching work and similar jobs the setting of concrete may be controlled in this way.

CHIMNEYS.

In an article upon the design of chemical works chimneys ‡ T. S. Clark draws attention to the use of an acid-proof mortar made from pure silica sand and silicate of soda.

There is a special type of silicate of soda on the market designed for the manufacture of acid-proof cements, and it is suggested that both brick and concrete chimneys could advantageously be coated with such an acid-resisting mixture.

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* Moyer—*Eng. Record* 62 (1910), 624.

† Dement—*Concrete*, 11 (1917), 52.

‡ Clark, *J. Ind. Eng. Chem.* 15, (1923), 227; *Chem. Trade J.* LXXII (1923), 355.

See also: Moyer—*Concrete*, IV (1910), 49; *Concrete-Cement Age*, IV, 1914; *Concrete*, XVI (1920), 279; *C.A.*, 1910, 2563.

WIND: AND ITS EFFECTS UPON STRUCTURES.

By H. F. CORNICK, M.C., A.M.Inst.C.E.

(*Concluded.*)

IND PRESSURE ON BUILDINGS.

effective wind pressure on buildings besides being influenced by their ape and form, largely dependent upon heir situation. In towns, care must be aken to note the general lay-out of the eighbouring structures, as it is possible or the disposition of the various buildings greatly to influence wind pressure: or example, the funnel effect of a street of houses. In many instances it is desirable to know both the positive pressure on the windward side and the negative pressure on the leeward side, especially in the case of long narrow buildings, and, as has been pointed out, in large buildings the uplifting effect will sometimes need to be taken into account. The height of a building is of great importance in deciding what wind pressure to allow for and the height above ground at which to take it as acting. As has been seen, the velocity and pressure of wind increase with the height from the ground, but we must not forget that the pressure on large areas is not relatively so great as on small ones. In general, the determination of wind stresses in buildings does not present much difficulty.

There have been some extraordinary effects due to tornadoes and cylones in America, and much damage has been done not only by the actual movement of air but due to the vacuum caused by the fall of barometric pressure. Walls of houses have fallen outwards, and floors, roofs and windows have been burst outwards owing to the fall in pressure outside: happily, however, we are not troubled with such phenomena in this country.

As regards the wind pressure to be allowed for in practice no hard and fast rules can be laid down. Each building must be taken on its own merits. In general, however, a pressure of 30 lb. per sq. ft. of exposed surface may be taken for all members supporting an area of 300 sq. ft. or less. This pressure might be diminished for areas in excess of this, and for buildings in sheltered positions, but no value less than 20 lb. should be used.

If the height of the building is more than 30 ft., the pressure of 30 lb. should be increased by 25 per cent. and the centre of pressure for overturning calculations taken at two-thirds the height of the building.

Wind pressure on roof surfaces should be taken as the normal component of a wind of 30 lb. per sq. ft. Some engineers, in addition to the wind load, provide against a snow load, but as snow and a violent hurricane are unlikely to be at a maximum together this seems unnecessary unless one or the other is reduced.

In some American States a horizontal pressure of 30 lb. is allowed for on the roof and sides for all buildings, and 20 lb. if the building is sheltered and does not exceed 30 ft. in height. A pressure of 20 lb. per sq. ft. is equal to a gale of about 75 miles per hour, and that of 30 lb. per sq. ft. to a hurricane which is not likely to be exceeded in this country in situations where commercial buildings are erected. As a matter of interest it may be noted that the gable wind screen on the roof of St. Pancras railway station was calculated to withstand a wind pressure of 40 lb. per sq. ft. In the case of several airship sheds erected in various parts of England during the war, wind pressure was taken as varying uniformly from zero at the ground level to 50 lb. per sq. ft. at the eaves, which were anything up to 100 ft. high.

SHIELDING EFFECT IN BRIDGES AND TRESTLE PIERS AND TOWERS.

There is considerable shielding effect of the leeward members of a frame structure, such as a bridge or similar erection, by those on the windward side, dependent mainly upon their distance apart.

Experiments by Sir Benjamin Baker and Dr. Stanton give us some interesting facts concerning this, which are very much in agreement. Dr. Stanton's figures for experiments on one flat circular plate placed behind another one with wind acting normal to the surfaces at varying distances apart are as follows:

Distance apart in diameters:	1.0	1.5	2.0	3.0	4.0	5.0
Coefficient for total pressure:	0.75	0.75	1.00	1.25	1.5	1.78

Similar results were obtained with square plates, but with rectangular bars the shielding effect was considerably less.

Dr. Stanton also carried out further experiments with larger latticed panels, and found that when the girders were spaced two depths apart with a flat plate between them, presumably to act as a floor, the wind pressure on the leeward girder was only 25 per cent. of that on the windward girder. The wind pressure on the leeward girder was much less when the panels were spaced one depth apart, being then only 15 per cent. With no deck-plate and the lattice girders at one depth apart the pressure on the leeward girder was 28 per cent. of that on the windward one.

These experiments were made on models with considerable bracing. In actual bridges the percentage of apertures will not be less than 50 per cent. of the total area of the girders—that is to say, each girder—and in all probability will average from 60 to 75 per cent. The shielding action will not be so great in actual practice on this account; moreover, in deck-spans if there is a deck plate, and in through spans, plate, and lattice, the floor cross-girders and railbearers, although these members increase the shielding action of the windward girders, will of themselves have a considerable wind resistance.

The determination of effective wind pressure on bridges is a subject which presents many difficulties, arising chiefly out of the irregular shape and construction of those other than plate-girder spans. Having given the wind pressure to allow for, we have in all bridges correctly to estimate its effective pressure on the main girders of a plated span, and the booms, struts, bracing, floor, etc., of a lattice span, all presenting probably different shapes and forms. Besides this we have, as has been seen in many cases, the shielding effect of the windward girders and double bars.

In plate-girder spans the main girders are not, strictly speaking, flat plates, owing to their flanges, which make the girders of a depressed cup-shape form; this adds materially to their resistance to wind pressure—as much as 10 per cent. has been estimated for this effect.

For ordinary moderate-span lattice-girder bridges it is the practice to estimate the wind resistance of the girders by calculating the area of the surfaces in side elevation and taking multiples of this area to include for the leeward semi-shielded girders.

It is not easy to lay down hard-and-fast rules for calculating areas and effective wind pressures on bridges. The following series of data has been tabulated, and may be taken as good practice for ordinary types. Much, however, will depend on the regulations in force and on the individual judgment of the engineer.

AREAS OF BRIDGES AND MULTIPLES FOR WIND PRESSURE.

(1) PLATE GIRDER DECK SPAN BRIDGES (WITH DECKING).

Single track; girders spaced 1–2 depths apart; area as seen in elevation, plus 10 per cent. for cup shape and 20 per cent. for elongated shape.

(1A) PLATE GIRDER DECK SPAN BRIDGES (WITHOUT DECKING).

As before; 1½ times area seen in elevation, plus additions as before. Wider spacing of main girders, multiple increased in proportion.

(2) PLATE GIRDER THROUGH BRIDGES (WITH DECKING).

Single track; girders spaced 2–3 depths apart; 1¼ times area seen in elevation, plus additions for shape as before.

(2A) PLATE GIRDER THROUGH BRIDGES (WITHOUT DECKING).

As before; 1½ times area seen in elevation, plus additions as before. Wider spacing of main girders, multiple increased in proportion.

(3) LATTICE GIRDER THROUGH SPAN BRIDGES (WITH DECKING).

Single track; girders spaced about 2 depths apart; 1½ times area seen in side elevation.

(3A) LATTICE GIRDER THROUGH SPAN BRIDGES (WITHOUT DECKING).

As before; 1¾ times area seen in side elevation; wider spacing of main girders, multiple increased in proportion.

Lattice girder deck spans as for lattice girder through spans.

We have so far assumed that the wind acts always normally to the axis of the bridge, but it is obvious this assumption is not always correct. When the wind acts at an angle to the axis of the bridge, while not making any great difference to the shielding effect of the windward girder, the shielding effect of double bars will be considerably reduced.

Considering Dr. Unwin's table (Table 6) it will be seen that the wind can vary 30 deg. from the normal to a surface without appreciably affecting the side pressure on that surface; moreover, according to the Table quoted, wind at angles of 20 and 10 deg. produces greater pressures at right angles to the surface than if the wind itself were acting normal to the surface. But with wind acting at angles of more than 30 deg. to the normal its effective pressure at right angles to the surface is reduced.

The question as to whether these decreased pressures on the semi-shielded bars would be more than in the case where the wind acted normally to the surface of the bars with others directly behind them will depend upon the distance apart of the bars and the angle at which the wind acts. In bridges of large dimensions, these relations would have to be considered.

The shielding effect of double bars in bridges where these occur should be taken account of as follows. Assume the wind acting at a slight angle and calculate the areas of the leeward bars unshielded by the windward bars at their distance apart. This additional area should be added to the total area of the windward girder as seen in side elevation.

The shielding action of double plates at 1 to 1½ diameters apart is three-quarters of the pressure on one plate; that is to say, the leeward plate decreases the negative pressure on the rear side of the windward plate. In the case of bars this effect is reduced, and we might substitute the value of 1 for the total pressure on both bars.

With wind at an angle this shielding effect is considerably reduced, and for bars spaced one width apart and wind at 30 deg. the total pressure on the bars would probably amount to 1¼ times the pressure on a single bar. In most cases a wind at an angle will take effect upon the cross girders and floor bearers, and this additional area should be added to the total area of exposed surfaces of the bridge; that is to say, after the area of the windward girder has been increased by the multiple for exposed surface of the leeward girder for total exposed surface.

In bridges of large magnitude a somewhat different and more exact computation of wind resistance is often necessary. The form of the compression and tension booms, struts, and ties, and their relative size, may merit separate consideration. The total of their separate resistances will be the total wind resistance of the bridge.

OVERTURNING EFFECT IN BRIDGES.

The overturning effect of wind pressure on bridges is often of much greater consequence than the stresses induced in the members themselves. Particularly is this the case in small loaded spans. The degree of this action is dependent upon the position of the bearings in relation to the centre of pressure at which the wind acts, and the width between the main girders, being greatest in the case of a single-track deck-span in which the bearings of the bridge are situated at the ends of the bottom flanges.

It is customary to specify a reduced wind pressure for a loaded span, as it is doubtless very improbable that a train will be travelling at its maximum speed during the maximum wind of a heavy storm; in other words, it is not likely that maximum impact will occur at the same time as maximum wind-pressure stresses. In the case of large bridges the wind load on rolling-stock will not bear a very large proportion to the wind load on the structure, and in bridges of the size of the Forth and Quebec bridges the wind pressure on the train will be almost, if not quite, negligible.

A method of dealing with the case in respect of moderate-size bridges is as follows:

Assume the wind blowing at a slight angle to the axis of the bridge so as to take effect on the exposed areas of the floor and of both windward and leeward girders and of any semi-shielded bars. Then, with a train on the bridge, take the wind blowing in the same direction on the train and on the exposed surfaces of the bridge only, i.e., those parts unshielded by the train, but with a reduced wind pressure. The pressure on the train to be taken as a live load, to which a percentage, depending upon the length of the span, should be added for impact. The maximum stresses resulting from either condition to be taken in determining the necessary sectional area of the parts required for resisting wind pressure.

The wind pressure on arched bridges is not often a serious matter, as owing to their shape they present little surface at their centres, where the wind has its greatest effect.

There is in many bridges an uplifting effect of the wind. In situations abroad where wind velocities are higher and cyclones are prevalent this may be of serious moment. The uplifting force in these cases is not so much due to the horizontal wind as to the sudden local changes in barometric pressure. In America there is on record a bridge of 100-ft. span having been lifted up bodily off its bearings and dropped into the middle of the river. It is well-nigh impossible to estimate the forces in such circumstances, and all we can do is to pay particular attention to the end fastening of the girders to the abutments. In this country all bridges which are secure against overturning will be reasonably safe against any uplifting force due to shape and wind pressure.

WIND PRESSURE ON TRESTLES AND TOWERS.

The wind pressure on the steelwork of trestles and towers and similar structures is not of serious moment compared with the overturning effect of the wind due to the girders of the bridge, and the rolling stock upon them, or the tanks or other structures supported upon them. The uplifting of the feet of the piers or stanchions must be calculated in order to provide ample strength in the holding-down bolts and sufficient weight in the foundations to counteract this tendency. Many engineers do not allow anything for the shielding effect of the windward members of trestles and the like; in any case the effective area taken for computing wind resistance should not be less than 1¾ times the total area in side elevation.

DYNAMIC ACTION OF WIND.

There is considerable divergence of opinion as to the advisability of allowing a percentage on the wind pressure for the effects of impact. Perhaps in the first case we require more data in regard to the " space area " of gusts and squalls, and also the length of time during which gusts last. We all know that most destruction and havoc is wrought by the wind when it comes in violent gusts and squalls. The maximum stress in the girders of a bridge due to a gust of wind would undoubtedly only be reached if the increase of pressure were instantaneously applied and if it endured long enough to produce one complete vibration of the structure, but we need not necessarily allow 100 per cent. impact.

The oscillations of a tall tree in gusty weather show that its maximum deflection is greater than that due to the actual wind pressure at any moment, and it is conceivable that a series of gusts might synchronise with what might be called the oscillation period of the bridge, so as to sway it, keeping time with its natural period of vibration, in which case very much greater stresses are likely to be involved, even though the wind has never at any time either reached the maximum velocity or pressure specified, or, on the other hand, been instantaneously applied. It seems, therefore, very reasonable that as we allow for the effects of variations in stress which are due to changes of load it would be equally correct to make at least some allowance for sudden changes of wind loads or pressure.

In the ordinary course of bridge design the wind load on rolling-stock passing over a bridge is treated as live load, but this, as has been seen, only affects comparatively small bridges to any degree. The maximum pressures of wind taken for bridge design in this country have without doubt amply covered the dynamic effects of wind, but if we are to reduce our wind pressure for bridge and other structural calculations the effect of dynamic action of the wind needs some consideration.

INERTIA OF STRUCTURES.

The heavier a structure is the safer it is against overturning, but the effects of impact—that is to say, the stresses induced—are present whatever the weight of the structure, and, as we have seen, appear to warrant allowances being made for them. The inertia of the mass of a building or other structure or a bridge, however, is a great asset against any overturning tendency.

We can easily calculate the steady wind pressure necessary to overturn any structure, and a sudden and momentary increase in velocity of the wind would have no appreciable effect upon the result —there is more than the weight of the structure to be overcome, there is its inertia. This fact may be illustrated by the way in which windows of buildings have sometimes been blown in when by calculation the building itself should have been overthrown first.

Another fact to which it may be desirable to draw attention is that a large structure is more stable than a small one of a similar shape and nature. In other words, it takes a greater pressure of wind to overturn a large structure than it does a small one of the same shape and density, if the structures rely only upon their weights to resist overturning. This statement is, of course, approximate only, and is dependent upon the material not being ruptured; moreover, it ignores the fact already noted that wind pressure is less at or near the ground than at a height, also that wind pressure on a large area is relatively smaller than on a small area. Roughly, however, the statement is true and may explain, as it has been put, why an insect has a comparatively large leg base and a large quadruped a proportionately small one.

CONCLUSIONS.

While observations and research have during recent years provided a great deal of useful data, by which previous practice has been to a large extent modified, and the design of structure to withstand wind pressure has been based upon correct principles, it will be obvious that any attempt to specify a standard wind pressure without regard to the size, shape, or form of the area upon which it acts may give results which may be insufficient for one structure and anything up to 100 per cent. too high for one of another form and construction. These remarks do not apply to so great an extent to buildings of steel (cased in with sheeting or brickwork), brick, and reinforced concrete, which are usually of more or less the same form. On the other hand, railway and highway bridges in particular, pit-head gear, coal and ore bunkers, cranes, gantries, water towers, and other industrial structures, are of so diverse a construction and form that much must be left to the individual judgment of the engineer or designer, and any attempt at standardisation unless very thorough and far-reaching may only lead to disastrous results.

A maximum wind pressure resulting from a velocity of wind of about 100 miles per hour, being the maximum velocity recorded in this country with any certainty, might be adopted as a standard for a certain area of exposed surface. We have then, at any rate, a definite starting point for calculation of resultant wind pressure on structures, and this pressure should only be modified by such considerations as the size of the exposed surface, sheltered situation, and height above ground level.

We have, then, given values to the two first desiderata of wind pressure, and it only remains to formulate, as far as practicable, standard methods of calculation for the effective wind resistance of structures and parts of structures of various shapes and forms. There is no doubt that with a wind pressure of 56 lb. per sq. ft. the somewhat rough and ready methods of calculation of effective wind resistance were as accurate as need be, but if a reduced pressure is adopted, more in accord with a wind pressure which it is possible may occur, some greater degree of accuracy will be needed in computing the effective wind resistance of complicated structures of whatever construction.

Some further experiments on the separate members of existing bridges and other structures of various types and forms, and in varying situations would seem to be necessary in order to provide factors for modifying the given wind pressure on a flat plate of given area for use on structures and portions of structures of other sizes, shapes, and forms. Such experiments would not be difficult of operation and would bestow untold benefit on the community at large, for the hurried generalisation in the adoption of an arbitrary wind pressure and the methods of its application to structures are often the means of spending vast sums of money unnecessarily.

In the absence of more detailed information as to wind pressure effects, the engineer must again fall back upon that much-abused panacea—factor of safety—having perhaps a shrewd idea that the extra material supplied may possibly only be so much dead weight and may never be required.

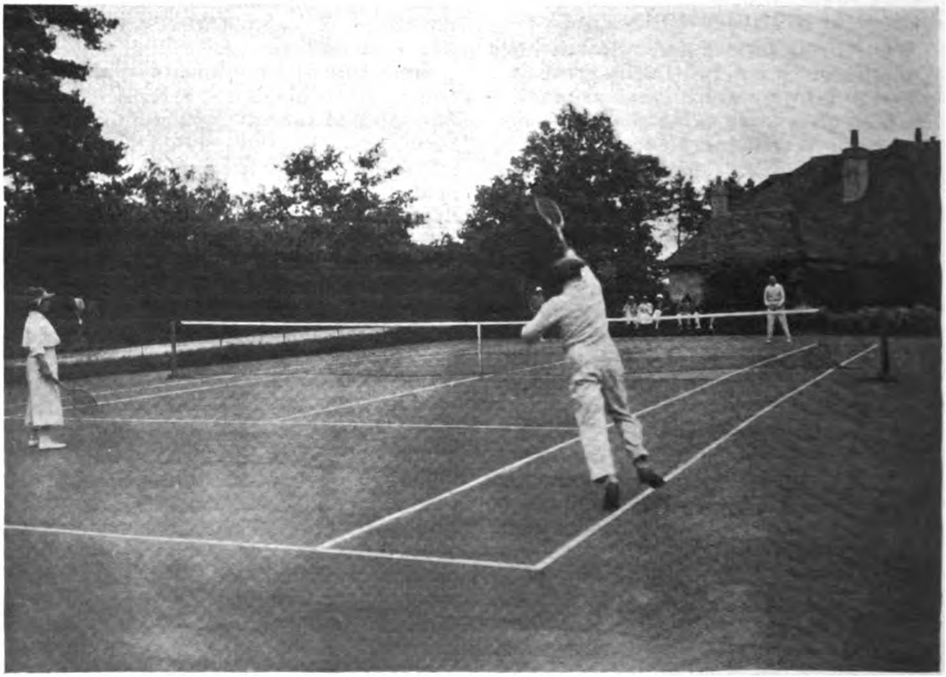
It is to be hoped that, although much has recently been done, research on the lines above suggested will be continued and elaborated, and the results and conclusions thereon tabulated and made public for practical use.

CONCRETE TENNIS COURTS.

THE growing popularity of the game of lawn tennis has led to the introduction of various forms of hard courts with a view to rendering play possible when it would be impracticable on a grass court. For this purpose concrete is receiving considerable attention on both sides of the Atlantic, since this form of court permits play practically all the year round.

The principal objection to grass and some types of hard court is that they are expensive to keep in good condition. The concrete tennis court possesses many advantages. It is not affected by rain, and the surface drains so quickly that play can be resumed a few minutes after a shower. The concrete court does not become dusty, is not damaged by the shoes of the players, and is not affected by frost or heat. The surface being even and dependable without being slippery, a faster and more accurate game is ensured. This is the almost unanimous opinion of those who play regularly on concrete courts. The principal advantage, however, is that concrete courts are economical, because being always in good condition there is no upkeep.

As a guide to those who are desirous of using this form of construction for their tennis courts we give below a suggested specification. The court in our illustration, near Mortimer, Berks, was laid in accordance with this specification. The surface was coloured green, with a special preparation, and the total cost was £160.



CONCRETE TENNIS COURT NEAR MORTIMER, BERKS.

Suggested Specification for Concrete Tennis Courts.

Foundation.—The preparation of the foundation will necessarily vary with local conditions. Any soft or weak places should be excavated and filled with hard stone or other suitable material, and the whole of the surface should be consolidated as much as possible. Before concreting, every vestige of vegetation should be cleared away and the foundation soaked with water.

Proportions.—The proportions of the material employed should be as follows :—

Base.—1 part cement, 2½ sand, and 4 coarse material (from ¼-in. to 1-in. in size).

Upper Course.—1 part cement to 3 parts fine aggregate (¼-in. downwards) mixed fairly dry and well tamped or rammed.

Thickness.—The thickness of the base shall be 3 in. and on this a wearing course of 1 in. in thickness shall be laid.

Placing.—The concrete for the surface shall be placed upon that for the foundation within twenty minutes after the latter has been laid and before the base has begun to set.

Surface.—To allow any water to run off the court the surface should be made to slope 1-in. from the net line to the back of the court. Provision should be made for the sockets of the posts. The surface should be finished with a wooden float and not a steel trowel, in order to avoid slipperiness.

Joints.—A joint should be made at the net line ½-in. wide filled in with a strip of tarred felt or bituminous material.

Colour.—The concrete for the surface may be coloured a suitable shade by the addition of mineral pigments. If a red tile effect is required the addition of approximately 14 parts red oxide of iron to every 86 parts of Portland cement will produce a good shade. For green approximately 10 parts of oxide of chromium should be mixed with every 90 parts of Portland cement. The exact shade can only be found by experiment, and the above proportions merely serve as a guide. It should be remembered that coloured concrete is lighter when dry than wet. Another method of colouring is by the application of a coat of paint specially made for painting on freshly-laid concrete. This paint is supplied by several firms.

Provision for Permanent Lines.—When the court is being constructed channels should be made to form the lines, in which concrete made of white marble chips with plenty of fine material in it (formed by crushing) mixed with white Portland cement should be filled. The proportions should be 2 parts marble to 1 part white Portland cement. This will give the purest white possible. The nearest approach to this is obtained by using crushed Portland stone as the aggregate. If neither of these aggregates is available the channels should be filled with mortar made of 1 part white cement to 1 part sand. The mortar should not be too wet—just a good plastic consistency.

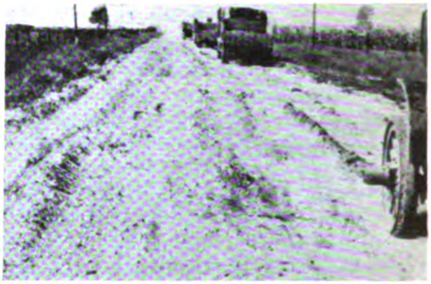
Protection.—As soon as the concrete is hard enough to withstand pitting it should be sprayed with water, and as soon after as may be done without damaging the surface the concrete should be covered with about 2 in. of wet sand, earth, or other material, which will give equally good protection and keep wet for a week or ten days, during which time the court should not be used.

Note.—Reinforcement is not considered necessary for a tennis court, but should it be used the position suggested is on the top of the lower course. If reinforcement is used it need only be of a very light description.

The British Engineers' Association.

THE eleventh annual general meeting of the British Engineers' Association was held recently and Mr. E. W. Petter (Chairman of Messrs. Petters, Ltd., of Yeovil) was elected President, in succession to Mr. Nevile G. Gwynne, C.B.E., who had occupied that position for the past two years. The organisation of the Shipbuilding, Marine, Mechanical and General Engineering Section of The British Empire Exhibition, Wembley Park, has been undertaken by the Association, and it was reported that already over 75 per cent. of the space to be devoted to these exhibits had been allotted through the Association. Mr. H. J. Ward, of Messrs. J. & E. Hall (Dartford), was elected Chairman of the Executive Committee in the place of Mr. Petter.

ALL-CONCRETE SECTION AFTER BEING TRAVERSED 10,000 TIMES.



MACADAM (ASPHALTE SURFACE) SECTION AFTER BEING TRAVERSED 3,000 TIMES.

THE BATES ROAD TEST.

WE give on this and the following pages results of what is undoubtedly the most exhaustive test ever undertaken in order to gain a definite knowledge of the wearing properties of various kinds of roads when subjected to modern traffic. The test road consisted of over sixty-three sections, each 18 ft. wide and from 100 ft. to 250 ft. in length, laid end to end, so that they made a continuous straight pavement two miles long. Twenty-two of these sections were of brick, seventeen of asphalte, and twenty-four of concrete. In each type of road the sections varied from those so thin that they were sure to break under the lighter test loads to sections which were considered strong enough to support successfully the heaviest loads permitted by the United States traffic laws. All the materials used in the different sections conformed to the standards required by the Illinois State specification, which are very high.

Over this road a fleet of army trucks made 23,000 round trips. Beginning with the bare chassis, loads were increased in 2,000-lb. increments until a gross load of 12¼ tons was reached. The greatest traffic imposed upon any section was 377,460 tons, of which 52·6 per cent. was the legal limit of 4 tons per rear wheel.

Many of the early failures are what one would naturally expect, but it is gratifying to see that of the thirteen sections which survived the test without failing ten were all-concrete roads. The other three types which did not fail were asphalte on a 7-in. concrete base and 3-in. brick on a 5½-in. concrete base. The concrete roads which failed at a gross load of 14,700 lb. were none of them more than 5 in. thick. Three single-course reinforced all-concrete roads, 5 in., 6 in. and 7 in. thick, failed at a load of 21,900 lb. One reinforced concrete road 5 in. thick and one plain concrete road 5 in. thick, both laid in one course, failed at 28,420 lb.

It is, however, in the sections which survived the test that the chief interest centres, and these seem to provide some well-defined indications as to the relative merits of the systems in use. For example, of the ten all-concrete sections which survived the test, seven were reinforced. The reinforced sections varied in thickness from 5 in. to 7 in., while the unreinforced sections were 8 in. and 9 in. thick. A system of circumferential reinforcement was used in all the reinforced sections which survived.

The concrete in all the concrete roads was a 3½ : 2 : 1 mixture.

FIRST FAILURES: GROSS LOAD, 9,500 LB.; LOAD ON EACH REAR WHEEL, 2,500 LB.; TIMES TRAVERSED, 1,000.

TYPE.	SURFACE.	BASE.	CUSHION. In.	TOTAL THICKNESS. In.
ASPHALTE ON GRAVEL	2 in. "Tapeka"	4 in. "Novoculite"	—	6
MONOLITHIC BRICK	3 in. Brick	2 in. 1 : 2 : 3½ Concrete	—	5
CONCRETE ("CEMITE") CEMENT . .	4 in. 1 : 2 : 3½	—	—	4

SECOND FAILURES: GROSS LOAD, 11,300 LB.; LOAD ON EACH REAR WHEEL, 3,500 LB.; TIMES TRAVERSED, 3,650.

TYPE.	SURFACE.	BASE.	CUSHION. In.	TOTAL THICKNESS. In.
BITUMEN FILLED BRICK ON MACADAM	3 in. (with lug)	4 in. Macadam	2	9
,, ,, ,, ,,	3 in. (lugless)	4 in. ,,	2	9
,, ,, ,, ,,	4 in. (with lug)	4 in. ,,	2	10
,, ,, ,, ,,	4 in. (lugless)	4 in. ,,	2	10
,, ,, ,, ,,	4 in. (with lug)	8 in. ,,	1	13
,, ,, ,, ,,	3 in. (lugless)	8 in. ,,	2	13
ASPHALTE ON MACADAM	2 in. "Tapeka"	8 in. ,,	—	10
,, ,, ...	2 in. "Tapeka"	4 in. ,,	—	6
BITUMEN FILLED BRICK ON CONCRETE	4 in. (with lug)	4 in. 1 : 2 : 3½ Concrete	1	9
,, ,, ,, ,,	3 in. (with lug)	4½ in. 1 : 2 : 3½ ,,	1	8½
,, ,, ,, ,,	3 in. (with lug)	4½ in. 1 : 3 : 5 ,,	1	8½
,, ,, ,, ,,	3 in. (with lug)	3½ in. 1 : 2 : 3½ ,,	1	7½
,, ,, ,, ,,	3 in. (with lug)	3½ in. 1 : 3 : 5 ,,	1	7½
CONCRETE (2½% CALCIUM CHLORIDE)	5 in. (1 : 2 : 3½)	—	—	5
,, (7½% HYDRATED LIME) .	4 in. (1 : 2 : 3½)	—	—	4
,, ...	4 in. (1 : 2 : 3½)	—	—	4

THIRD FAILURES: GROSS LOAD, 14,700 LB.; LOAD ON EACH REAR WHEEL, 4,500 LB.; TIMES TRAVERSED, 3,000.

TYPE.	SURFACE.	BASE.	CUSHION. In.	TOTAL THICKNESS. In.
ASPHALTE ON MACADAM	2 in. "Tapeka"	10 in. Macadam	—	12
BITUMEN FILLED BRICK ON CONCRETE	3 in. (lugless)	4½ in. 1 : 2 : 3½ Concrete	1	8½
SEMI-MONO BRICK	3 in. Brick	2 in. 1 : 2 : 3½ ,,	¾	5¾
MONOLITHIC BRICK	4 in. ,,	2 in. 1 : 2 : 3½ ,,	—	6
,, ,, ...	3 in. ,,	3 in. 1 : 3 : 5 ,,	—	6
,, ,, ...	3 in. ,,	3 in. 1 : 2 : 3½ ,,	—	6
,, ,, ...	4 in. ,,	3 in. 1 : 2 : 3½ ,,	—	7
CONCRETE	5 in. 1 : 2 : 3½	—	—	5
CONCRETE (4% CALCIUM CHLORIDE) .	5 in. 1 : 2 : 3½	—	—	5
,, ("CIMENT")	5 in. 1 : 2 : 3½	—	—	5
,, (2½% CALCIUM CHLORIDE)	4 in. 1 : 2 : 3½	—	—	4
,, ...	4 in. 1 : 2 : 3½	—	—	4

FOURTH FAILURES: GROSS LOAD, 18,300 LB.; LOAD ON EACH REAR WHEEL, 5,500 LB; TIMES TRAVERSED, 3,000.

TYPE.	SURFACE.	BASE.	CUSHION. In.	TOTAL THICKNESS. In.
ASPHALTE ON MACADAM	2 in. "Tapeka"	6 in. Macadam	—	8
ASPHALTE ON CONCRETE	2 in. "Tapeka"	4 in. 1 : 3 : 5 Concrete	—	6
,, ,, ,, ...	1½ in. "Tapeka"; 1½ in. Binder	4 in. 1 : 3 : 5 ,,	—	7
,, ,, ,, ...	2 in. "Tapeka"	4 in. 1 : 2 : 3½ ,,	—	6
,, ,, ,, ...	1½ in. "Tapeka"; 1½ in. Binder	4 in. 1 : 2 : 3½ ,,	—	7
,, ,, ,, ...	Ditto	5 in. 1 : 2 : 3½ ,,	—	8
MONOLITHIC BRICK	4 in. Brick	4 in. 1 : 3 : 5 ,,	—	8
CONCRETE (PLAIN)	5 in. 1 : 2 : 3½	—	—	5
,, ,, ...	4 in. 1 : 2 : 3½	4 in. Macadam	—	8

FIFTH FAILURES: Gross Load, 21,900 lb.; Load on each Rear Wheel, 6,500 lb.; Times Traversed, 3,000.

Type.	Surface.	Base.	Cushion. In.	Total Thickness. In.
Asphalte on Macadam	1½ in. " Tapeka "; 1½ in. Binder	5 in. Macadam	—	8
Asphalte on Concrete	2 in. " Tapeka "	5 in. 1 : 3 : 5 Concrete	—	7
Bitumen Filled Brick on Concrete	3 in. (lugless)	4½ in. 1 : 3 : 5 ,,	1	8½
,, ,, ,, ,, ,,	3 in. (lugless)	3½ in. 1 : 3 : 5 ,,	1	7½
Concrete (Plain)	7 in (1 : 2 : 3½)	—	—	7
†Concrete (Circumf. Reinf.) . .	6 in. (1 : 2 : 3½)	—	—	6
Concrete (Wire Mesh Reinf.) . .	5 in. (1 : 2 : 3½)	—	—	5

SIXTH FAILURES: Gross Load, 28,420 lb.; Load on each Rear Wheel, 8,000 lb.; Times Traversed, 10,000.

Type.	Surface.	Base.	Cushion. In.	Total Thickness. In.
Asphalte on Concrete	2 in. " Tapeka "	5 in. 1 : 2 : 3½ Concrete	–	7
,, ,, ,, ...	2 in. ,,	6 in. 1 : 3 : 5 ,,	—	8
,, ,, ,, ...	2 in. ,,	6 in. 1 : 2 : 3½ ,,	—	8
Bitumen Filled Brick on Concrete	3 in. (with lug)	6½ in. 1 : 2 : 3½ ,,	9	10½
,, ,, ,, ,, ,,	3 in. (lugless)	6½ in. 1 : 2 : 3½ ,,	1	10½
,, ,, ,, ,, ,,	3 in. (with lug)	5½ in. 1 : 2 : 3½ ,,	1	9½
,, ,, ,, ,, ,,	4 in. (lugless)	4 in. 1 : 2 : 3½ ,,	1	9
,, ,, ,, ,, ,,	3 in. (lugless)	3½ in. 1 : 2 : 3½ ,,	1	7½
Concrete (Wire Mesh Reinf.) .	5 in. (1 : 2 : 3½)	—	—	6
Concrete (Plain)	5 in. (1 : 2 : 3½)	—	—	6

NO DECISIVE FAILURES:

Type.	Surface.	Base.	Cushion. In.	Total Thickness. In.
Asphalte on Concrete	2 in. " Tapeka "	7 in. 1 : 2 : 3½ Concrete	—	9
,, ,, ,, ...	1½ in. " Tapeka "; 1½ in. Binder	8 in. 1 : 2 : 3½ ,,	—	11
Bitumen Filled Brick on Concrete	3 in. (lugless)	5½ in. 1 : 2 : 3½ ,,	1	9½
Concrete (Plain)	9 in. (1 : 2 : 3½)	—	—	9
†Concrete (Plain)	8 in. (1 : 2 : 3½)	—	—	8
Concrete (Plain)	8 in. (1 : 2 : 3½)	—	—	8
†Concrete (Plain)	7 in. (1 : 2 : 3½)	—	—	7
†Concrete (Circumf. Reinf.) . .	6 in. (1 : 2 : 3½)	—	—	6
,, , ,, . .	6 in. (1 : 2 : 3½)	—	--	6
† ,, ,, ,, . .	5 in. (1 : 2 : 3½)	—	—	5
† ,, ,, ,, . .	5 in. (1 : 2 : 3½)	--	—	5
* ,, (Plain)	7 in. (1 : 2 : 3½ gravel)	—	—	7
* ,, (Plain)	7 in. (1 : 2 : 3½ limestone)	—	—	7

* Longitudinal joint full length of section; ⅝ in. deformed bars 5 ft. long at 10 ft. centres across longitudinal joint; continuous ¾ in. plain round painted bar 6 in. from each outside edge of slab.

† Transverse joints every 25 ft. and a longitudinal joint the full length of the sections.

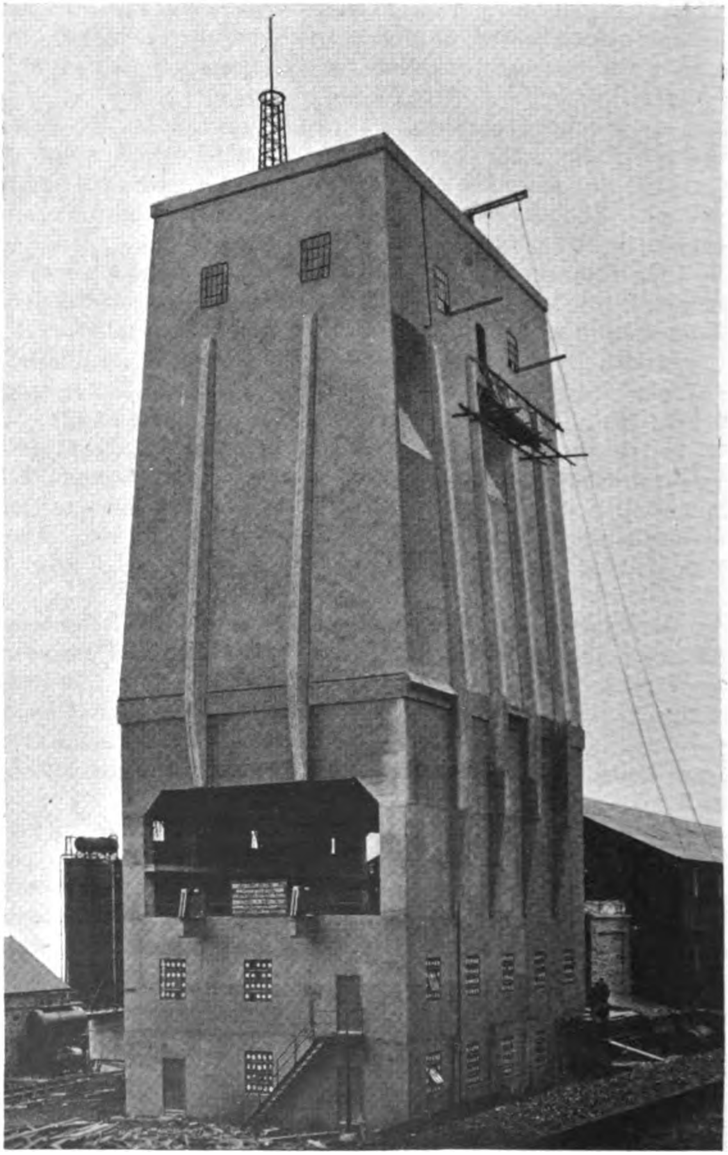
"CONCRETE ROADS."

THE necessity for improved methods of road construction to withstand the wear and tear of the fast and heavy traffic of the present day has led to a considerable demand for information on the all-concrete road, which, having been tried out in America during the past eighteen years, is now being adopted to an increasing extent in this country.

In 1920 we published our book entitled "Concrete Roads," which had a very cordial reception in the technical press, and for which there has been a steady sale. The first edition is now exhausted, and we are therefore publishing a second edition. The new edition has been brought right up-to-date to include current practice, and the contents are tabulated so as to make reference as simple as possible. The chapters include: Concrete Roads in England, Scotland, Wales and Ireland, with the latest Official Reports as to their condition, Weight of Traffic, Maintenance Cost, etc.; Concrete Roads in Industrial Works; Concrete Roads in Australia, New Zealand, Canada, and America, with latest reports; The International Road Congress at Seville, May, 1923—Extracts from Reports and the Resolutions; The Bates Tests of Concrete and other Roads; Mechanical Appliances used in the Construction of Concrete Roads; Concrete Roads and the various Reinforcements used; Concrete Kerbs and Channelling; A suggested form of Specification for the making of Concrete Roads; Concrete Road Accessories (Pavements, Fence Posts, Telegraph and Lamp Posts, etc.); Useful Hints and Data for making Concrete and the Choice of Aggregates; Press and other Opinions on Concrete Roads; etc.

The extraordinary headway made by this type of road is forcibly brought home in this new edition, which shows that within the past four years the number of all-concrete roads laid in this country has increased by no less than 900 per cent., and that the number under construction at the present time is greater than ever before. In addition to the public roads, a large number of works' roads are now being laid in unsurfaced concrete, and these are also fully dealt with in the book.

The book is well illustrated throughout, and the particulars of the methods of construction used by different Borough Surveyors should be particularly valuable to those about to lay new roads—the volume is, indeed, the only publication in which this information has been collected together in a convenient form. The book will be ready in a few days (price 5s. net), and orders may now be placed with the Trade Manager, Concrete Publications, Ltd., No. 4, Catherine Street, Aldwych, London, W.C.1, by filling in and posting the form on p. 498.



NEW REINFORCED CONCRETE COAL STORAGE BUNKER AT CONSETT: FIG. 1. (*See page* 465.)

REINFORCED CONCRETE COAL STORAGE BUNKER AT CONSETT.

THE reinforced concrete coal storage bunker illustrated herewith has recently been completed at Consett, Co. Durham, by Messrs. Industrial Constructions, Ltd., of 44, Grosvenor-place, S.W.1, for Messrs. Coppee Co. (Gt. Britain), Ltd., as part of the first battery of Wilputte Coke Ovens (for which the Coppee Co. hold the patent rights) to be erected in this country, for the Consett Iron Co., Ltd. The bunker has a capacity of 2,000 tons.

The contract was secured in competition. The low figure submitted by Messrs. Industrial Constructions, Ltd., was made possible by the general lines of the structure being decided, in collaboration with an architect, so that not only was the cost of shuttering reduced to a minimum but a structure of pleasing proportions and appearance was produced.

The foundations were carried down to the soft rock which underlies the site, and these had to be deepened at one end, under the flues, on account of a seam of coal which makes its appearance at this point.

The design of the structure generally was made to comply with the rules recommended by the Royal Institute of British Architects.

Up to the level of the lorry-charging floor the work consists of simple column, beam, and slab construction. Above this level the work is perhaps more interesting from a designer's point of view on account of the frame construction which has been employed. Four of these frames are shown in the cross section B.B. (*Fig.* 2, p. 466), and two other frames in the longitudinal section. The four principal frames shown in the cross section consist of the vertical beams on either side, the deep beam which runs through at hopper level, and the smaller beam at superstructure floor level. These beams are hinged at the lorry charging floor level. The columns and beams to the superstructure walls and roof were also designed as frames, but fixed at the superstructure floor level.

On account of the high altitude and the severe winds which are experienced, especially during the winter months, special precautions had to be taken while constructing the bunker above hopper level. A heavy braced scaffolding was employed, part of which was carried through temporary holes in the walls in order to support the light scaffold outside. As the construction of this bunker went on uninterruptedly through the winter months, special precautions had to be taken to protect the concrete. On one occasion the temperature was as low as 26° F., and very rarely above 46° F. Concreting was stopped when the temperature fell below 34° F. The 6-in. concrete cubes which were made and subjected to the same conditions as the rest of the concrete proved that the measures taken to protect the concrete were satisfactory.

The reinforcement used throughout consisted of mild steel bars of British manufacture, having a tensile strength of 28 to 33 tons per sq. in. with an elongation of 20 per cent. in a length of 8 in. The reinforcement was bent cold, and when placed tied together with 16-gauge soft iron wire.

The large-scale detail (p. 468) shows the " hinge " which was employed at the base of the frame at loading-floor level. The design of these ribs incurred a good deal of work in the drawing office, and called for special precautionary measures to be taken during erection owing to the very exposed position of the site.

Throughout this structure mild steel bars were used for the reinforcement.

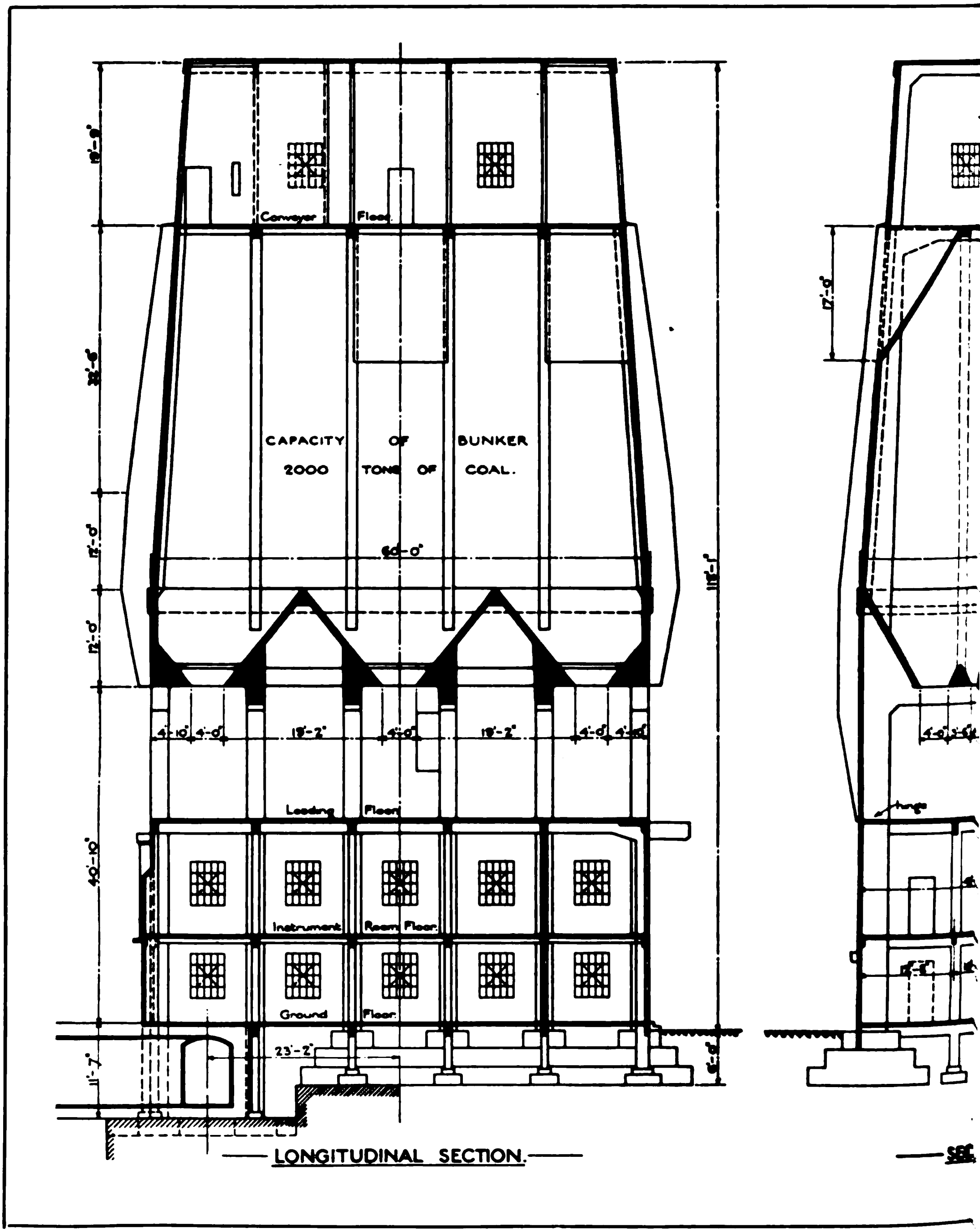
The concrete was composed of 4 parts of Scremerston pebbles to 2 parts of Scremerston sand to 1 part of Portland cement ground so as to leave not more than 3 per cent. on the 180 sieve.

The concrete cubes, which were tested from time to time, had an average crushing strength of about 2,250 lb. per sq. in. at an age of twenty-eight days, and 3,000 lb. per sq. in. at an age of three months.

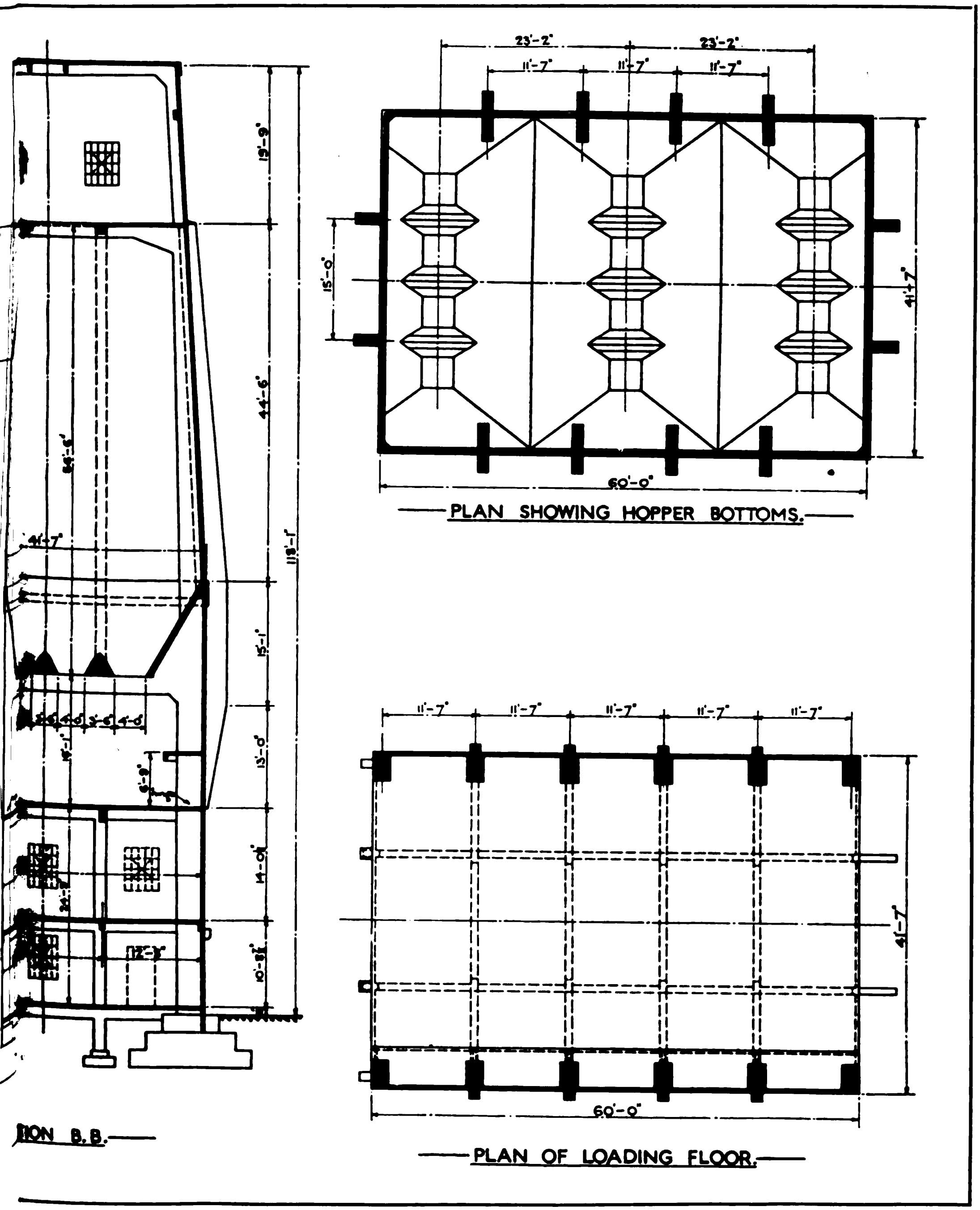
Upon completion the structure was rubbed down and cement-grout washed. No rendering was used in any part of the work except the floors, which were finished with ¾-in. granolithic.

An " Insley " 150 ft. (mast hoist) concrete placing plant was employed to hoist and distribute the concrete.

The whole of the concrete was machine mixed in a " Victoria " mixer, care being taken in the amount of water used.



REINFORCED CONCRETE COAL STORAGE BU

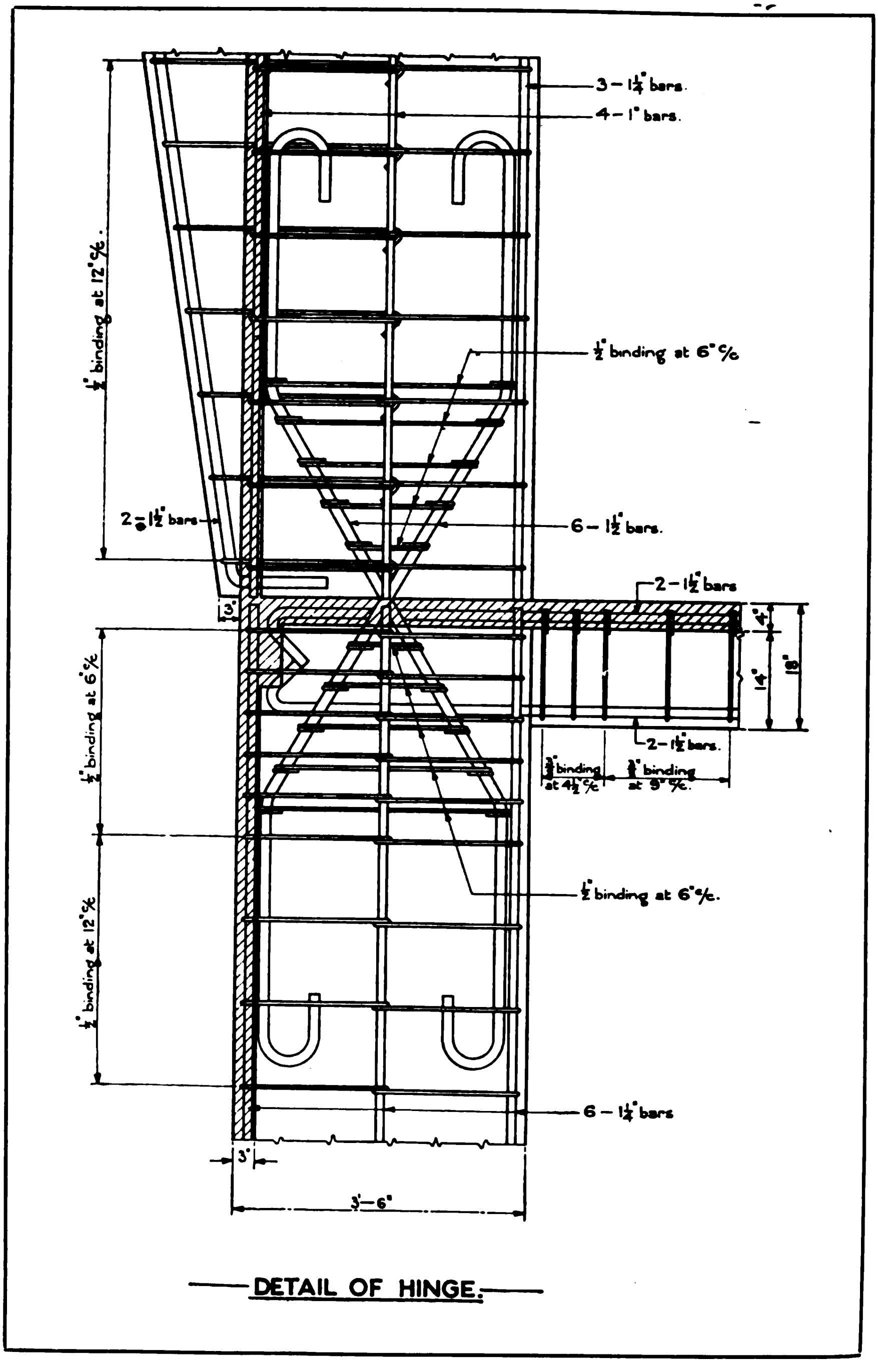


PLAN SHOWING HOPPER BOTTOMS.

PLAN OF LOADING FLOOR.

ION B.B.

NKER AT CONSETT: Fig. 2. (*See page* 465.)



New Reinforced Concrete Coal Storage Bunker at Consett: Fig. 3. (*See page* 465.)

CONCRETE CRANEWAY AT HAMBURG.

By CARL COMMENTZ (HAMBURG).

CONCRETE engineering is steadily extending its field of application in competition with steel construction, and a remarkable step in this development has recently been made by the erection of a large craneway for the plate stock of the Reiherstieg-Schiffswerfte, in Hamburg. The overhead travelling crane, as a moving construction, is entirely built in steel.

The purpose of the craneway, which has been designed and built by Messrs. Lange and Lahmann, is the discharging and transporting of ship plates and angles from railway wagons and barges.

The dimensions of the craneway were determined by the space at disposal and the purpose of the crane. The maximum height of the hook of the crane is 44½ ft. above ground level, and the height of the track is 37½ ft. above ground level. The distance between the two tracks is 98 ft. In order to be able to take material from barges, a cantilever of 23–26 ft. over the quay wall of the canal had to be added to the craneway. The supporting pillars are 39 ft. apart. Two cranes are used on the track with a single span between the ways and a maximum carrying capacity of 15 tons each. When loaded, the pressure on each wheel is 18·1 tons.

Originally it was intended to build the craneway of steel, but it was found that a reinforced concrete construction to carry the cranes would be much cheaper, as not only is the initial cost less but the maintenance expense is much cheaper as no painting is necessary. Also, the life of the concrete structure will probably be longer, as it is not affected by rust, and alteration of form by extension and contraction is less. However, as the length of the craneway is 458 ft. expansion and contraction had to be considered, and the connections between the supports and the craneway were built as flexible joints, which is, so far as I know, an entirely new feature in reinforced concrete construction. The general design was decided upon by the building firm in co-operation with its consulting engineer, Oberbaurat Scharff. Longitudinal rigidity was effected by fixing the craneway at the head of the middle pillar, while it is laid on all other pillars by flexible joints. The middle pillar is also supported by strong supports (*see illustration*). By choosing the middle pillar head as a fixed point, the necessary moving of the joints was limited to the smallest possible amount.

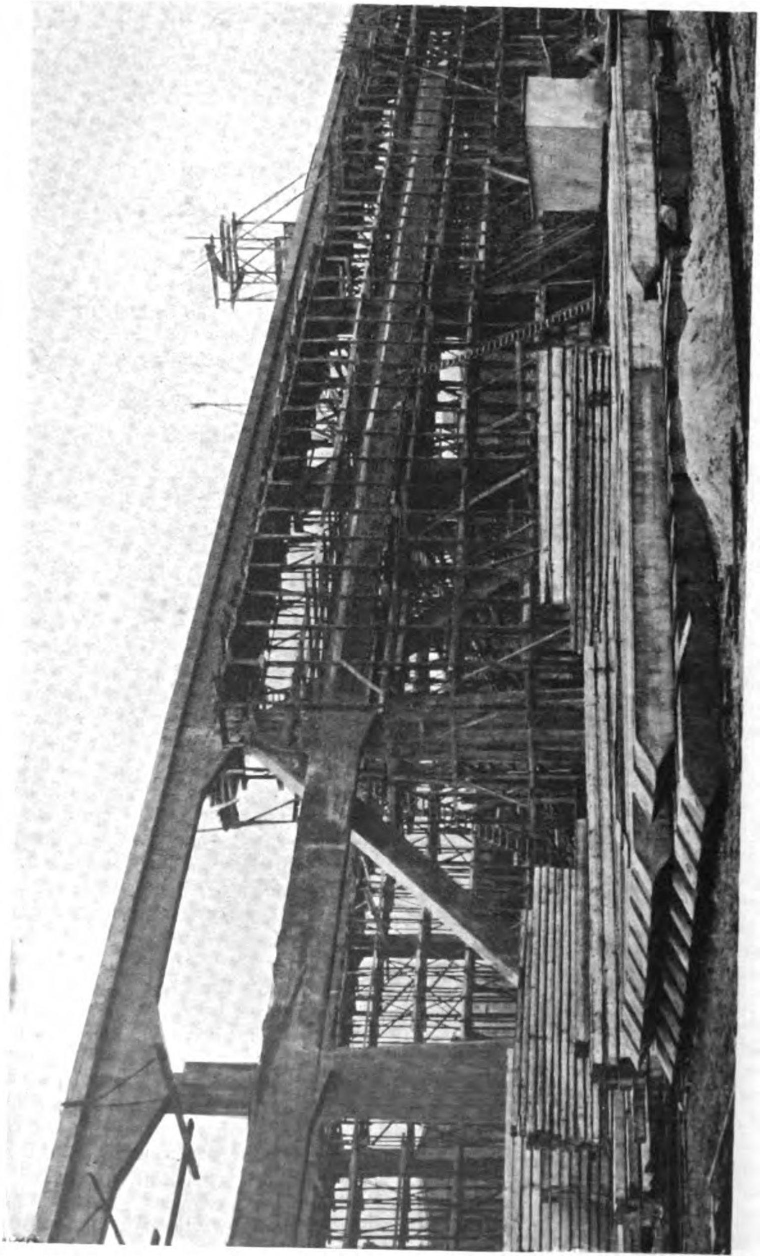
Bending moments from wind influence, brake, and starting forces required a considerable moment of inertia around the vertical axis, so that a T-section was chosen.

The foundation consists of reinforced concrete piles of a length of 29½ ft. with a square section of 1 ft. 1½ in. The southern row of pillars are on four piles each and the northern supports have six each. The pillars with the struts have two additional piles with an inclination of 2½ : 1 in order to take up the forces coming from the struts.

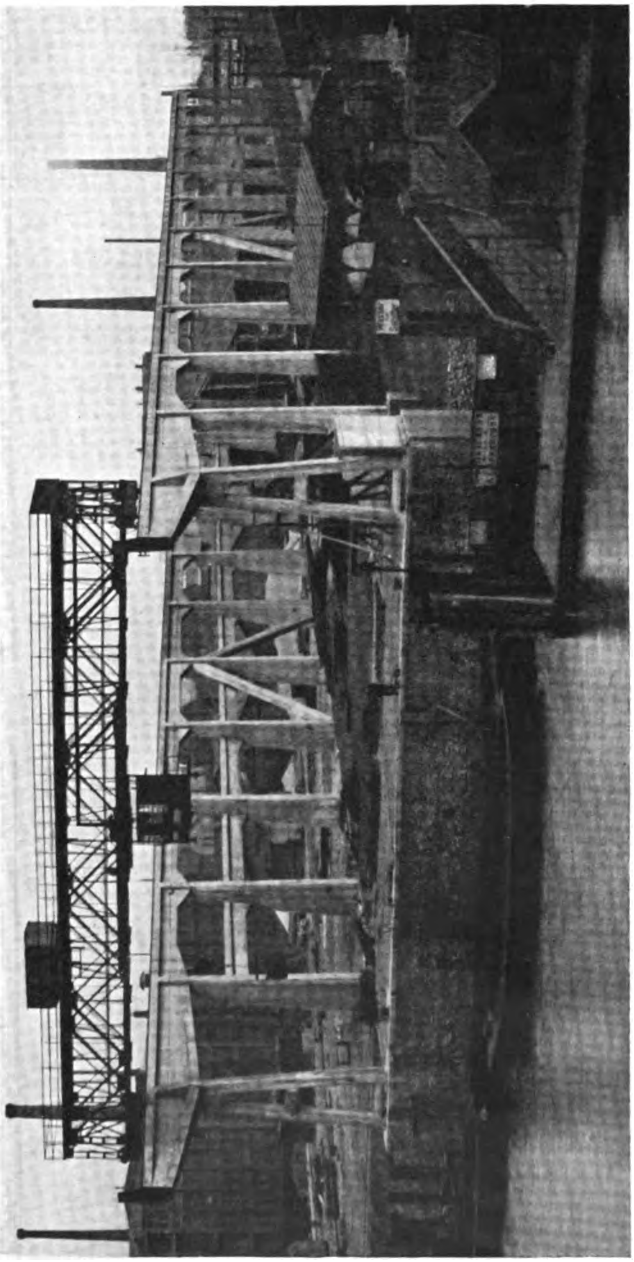
The flexible joints were built by simply crossing iron bars and pasteboard layers. At the two-leg support at the quay wall the surface of the joints was moulded round and the pasteboard replaced by a layer of lead two-fifths of an inch thick. Here the concrete was reinforced with special spiral reinforcement.

The reinforcement of the craneway proper has no special features. The cantilevers, which are exposed to considerable vertical and horizontal bending and shearing forces, longitudinal tension, pressure and torsion, required special measures. Apart from the longitudinal reinforcement, a complete net of vertical and horizontal hoops one-third of an inch in diameter was arranged at distances of 4 ft. 10 in. In the main construction the admissible tension and pressure of the reinforcement has been limited to 17,000 lb. per sq. in., while in order to have a somewhat greater factor of safety in the cantilever it was limited in this part to 1,400 lb. The concrete stresses have been limited to 560 or 850 lb. For the main construction the concrete was a 1 : 4 mix, and for the cantilever 1 : 3.

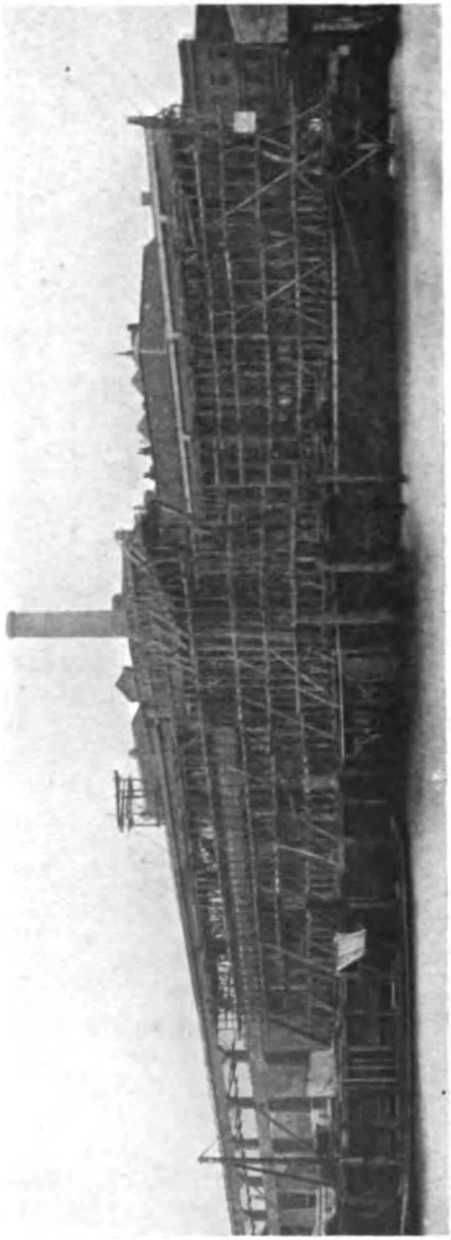
Work was commenced in March, 1921, and the foundations were completed in July. The concrete, which was mechanically mixed, was poured between shuttering, iron bars inserted, and tamped with small tampers; the settling was



Concrete Craneway at Hamburg.



Concrete Craneway at Hamburg.



Concrete Craneway at Hamburg.

also assisted by hammering the mould from the outside. All concrete work was completed by December, 1921, and in March, 1922, the craneway was ready for use.

When the craneway was ready a trial was made with a load of 19 tons and with the weight as close to one craneway as possible. The trial load was limited to this amount to prevent any cracking in the concrete. The crane was traversed successively from one span to another and kept at each point for ten minutes. The deflection was measured by a Griot recorder, and it was found that in the spans the deflection in no case amounted to more than 0·037 in., while the deflection of the cantilever was recorded at 0·287 in. On the removal of the load all deflection disappeared with the exception of the cantilever, where a difference of ·04 in. remained. The craneway has been in constant use and has given every satisfaction.

BOOK REVIEW.

Cements and Artificial Stones (a descriptive Catalogue of the Specimens in the Sedgwick Museum, Cambridge). By (the late) John Watson. Edited by R. H. Rastall, pp. xii + 131.

Cambridge: W. Heffer & Sons, Ltd. Price 6s. net.

As the sub-title of this small volume indicates, it is a catalogue of museum exhibits and is intended for the use of students of economic geology at Cambridge University. The book has, however, a wider interest than this on account of the historical and other notes attached to each group of specimens. Over 100 exhibits are described, comprising (1) raw materials for the manufacture of Portland cement, plaster of Paris and other cements; (2) specimens illustrating stages of manufacture of these cements; (3) briquettes and other specimens illustrating the testing of cements; and (4) specimens illustrating the uses of cements, including reinforced concrete, cement-asbestos goods, artificial stone, and plaster castings.

The historical notes are good and are written in a manner that adds interest thereto. The accidental occurrences that alternately helped and hindered the discovery and development of Portland cement are a striking contrast to the systematic and laborious research investigations of to-day. The remarks on concrete are sound, and the author could evidently have been classed as a concrete enthusiast. There is a great future for concrete if the university students of to-day can be imbued with the author's idea that " there is no branch of industry in the world that presents a wider field of research to the student of economic geology, applied chemistry, architecture, engineering, and other allied sections of natural science than that of cement, its manufacture and uses."

The author died in 1918, and it is unfortunate that when the manuscript was revised by the editor the opportunity was not taken to bring the matter up to date. For example, when the British Standard Specification is referred to on page 31 it is obviously that of 1915 and not the 1920 edition now current. Again, on page 19, referring to Portland cement, it is stated that the " fuel employed for calcining is usually gas coke," whereas the fact is that less than one-tenth of the cement made in England to-day is calcined with other than powdered coal.

The exhibits in the Sedgwick Museum could also be brought up to date with advantage, because it is misleading to the student to exhibit samples of " ordinary grinding cement ground to a fineness that leaves a residue of not more than 10 per cent. on a sieve having 5,800 meshes per sq. in." when practically the whole of the cement produced in Great Britain is ground to a fineness that leaves less than 1 per cent. residue on the sieve mentioned. Similarly, the testing briquettes (3 sand, 1 cement) that are exhibited have a tensile strength of 140 lb. per sq. in. at 7 days—a result which would warrant rejection of the cement nowadays when the best brands have tensile strengths of 400 lb. per sq. in. and upwards when mixed with 3 parts of standard sand. There are other statements not in accordance with modern ideas and to which exception might be taken. In a work intended for the use of students this is particularly regrettable, and is a serious blot on a book which might otherwise be recommended as of peculiar interest.

THE EFFECTS OF CEMENT CONTENTS ON CONCRETE.

At a meeting of the Institution of Structural Engineers, held on May 24, Dr. Oscar Faber, O.B.E., D.Sc., read a paper on "Experiments on Concrete of Different Cement Contents."

The paper was divided into Parts I, II and III, from which we take the following extracts :—

PART I.

In connection with some work requiring rich concrete, the lecturer some time ago wanted definite experimental results as to how the properties of concrete were affected by varying the cement contents, and could find only isolated and unrelated tests which, moreover, agreed but poorly with one another. He therefore determined to make a thorough and careful series of tests, in which cubes were made with 10 per cent., 20 per cent., 30 per cent., etc., up to 100 per cent. of cement, with careful measurements of weights of all the ingredients sufficient to furnish data as to weight of concrete, shrinkage of materials, quantities of material required, crushing strength, etc.

CONTROL OF WETNESS.

Such a series was made in August and September, 1922, and the quantity of water was left to the operator to find by trial, the resultant wetness to be controlled by a slump-test so as to obtain uniform wetness with all mixtures. It was found, however, that in spite of considerable care the results were very variable and bore strong internal evidence of unreliability, which was traced to variations in water content, which of course has a predominatory effect on strength of concrete. In other words, it was not found practicable, by means of leaving it to the discretion of the operator, assisted (or perhaps hampered) by a slump-test machine, to get the water contents sufficiently uniform in the various tests to obtain really uniformly varying results having strong internal evidence of reliability and consistency.

A second series was therefore undertaken in which the water content of each mixture was carefully regulated according to a formula which had been designed to give uniform wetness to both rich and poor mixtures. Even though it was subsequently found that this formula could be improved on slightly, the second series did produce the instructive and reliable results hoped for.

PROPORTIONING BY WEIGHT.

In order to obtain really accurate proportions of cement, sand, and ballast, it was found necessary to measure these by weight and not by volume, and it may be pointed out that the same cement can give variations in volume corresponding to a variation of from 90 to 60 lb. a cu. ft. depending solely on whether it be loosely packed or consolidated, and that a given measure filled with cement from a sack will give widely varying weights, and that clearly under such conditions no consistent results could be anticipated if the quantities were measured by volume. The same argument applies to sand and ballast, but to a much smaller extent.

SAND AND BALLAST.

In this series of experiments the ratio of sand to ballast was kept as 1 is to 2 throughout all the mixtures. The sand used was washed Ham River grit (a clean siliceous sand), and ¾ in. crushed Ham River ballast.

CEMENT PERCENTAGE.

Throughout the tests a 10 per cent. cement contents means that the weight of cement shall be 10 per cent. of the total weight of *dry materials* (cement, sand, and ballast) ; from which it follows that the sand and ballast would be 90 per cent., and since the sand is to be half the ballast, the sand would be 30 per cent., and the ballast 60 per cent. of the total weight of dry materials.

The cement percentage might have been taken as the percentage of the weight of *all materials* (cement, sand, ballast, and water), but this would conform less to usual parlance of the industry.

WATER CONTENTS.

The water content was calculated by taking 25 per cent. by weight of the

TABLE I.

Date of Test Mixing.	Proportions by Weight.	Weight of cement in grams.	Weight of ballast in grams.	Weight of sand in grams.	Weight of water in grams.	Weight of cubes after 24 hours in grams.	Weight of cubes after 28 days in grams.	Weight of residue after 24 hours in grams.	Temperature during mixing.	Crushing load in tons.	% Reduction in volume on volume of dry materials.
1	Neat cement	5,000	0	0	1,250	2,249	2,307	1,618	59°	62·6	17·74%
2						2,247	2,306			60·8	
1	9–1	4,500	333	167	1,150	2,263	2,315	1,466	60°	49·2*	19,9%
2						2,258	2,314			57·4	
1	8–2	4,000	667	333	1,050	2,287	2,347	1,281	61°	52·4	22·5%
2						2,296	2,357			52·6	
1	7–3	3,500	1,000	500	950	2,335	2,388	1,133	59°	38·2*	24·6%
2						2,334	2,387			48·8	
1	6–4	3,000	1,333	666	850	2,370	2,420	987	61°	43·8	26·5%
2						2,362	2,410			38·0	
1	5–5	2,500	1,667	833	750	2,415	2,458	830	59°	42·0	29·0%
2						2,407	2,443			41·8	
1	4–6	2,000	2,000	1,000	650	2,455	2,488	687	61°	41·6	31·0%
2						2,450	2,483			37·0	
1	3–7	1,500	2,333	1,167	550	2,466	2,495	544	58°	35·7	33·0%
2						2,465	2,500			36·8	
1	2–8	1,000	2,667	1,333	450	2,496	2,520	400	58°	30·8	34·2%
2						2,490	2,515			25·0	
1	1–9	500	3,000	1,500	350	2,504	2,518	251	58°	16·8	36·0%
2						2,500	2,512			17·4	
		27,500	15,000	7,499							

CALCULATION OF SHRINKAGE.

TABLE 1A.

% Cement.	100	90	80	70	60	50	40	30	20	10
Vol. cement	3460	3115	2770	2420	2075	1730	1385	7035	1690	345
„ sand	0	105	210	315	420	525	630	735	840	945
„ ballast	0	240	480	720	960	1200	1440	1680	1920	2160
Dry materials	3460	3460	3460	3455	3455	3455	3455	3450	3450	3450
Water	1250	1150	1050	950	850	750	650	550	450	350
All materials	4710	4610	4510	4405	4305	4205	4105	4000	3900	3800
Dry materials required to make 1 vol.	1.205	1.235	1.277	1.315	1.35	1,393	1.429	1.469	1.513	1.568
All materials required to make 1 vol.	1.645	1,647	1.666	1.675	1.68	1.70	1.71	1,71	1.71	1.726

cement plus 5 per cent. by weight of sand and ballast. It was found that this formula made the very rich specimens rather too dry and the very poor ones rather too wet, and that a better formula is 28 per cent. by weight of cement plus 4 per cent. by weight of sand and ballast.

Subsequent tests were made verifying that this did, in fact, give proper and uniform consistency to all mixtures, but the formula first given was used in the tests here described.

CEMENT.

The cement used gave the following results :—

Fineness.

Residue on	240 sieve	.	39.02 per cent.
,,	180 ,,	.	7.95 per cent.
,,	76 ,,	.	Nil.

Apparent Density.

Measured by allowing the cement to trickle slowly in fine streams through air into a measure by means of apparatus = 1.074.

[*Note* : This is a useful test as giving consistent densities readily, and in which the smallness of the density is a measure of the fineness of grinding and therefore of the quantity.]

This apparent density is not to be confused with that measured with the density of the particles, nor is it to be used in calculating proportions, as in practice the cement would always be more consolidated.

	Initial Set.	*Final Set.*
In Air	3 hr. 45 min.	8 hrs.
In Water	4 ,, 30 ,,	10 ,,

Expansion.

In Air . . . . 2 mm.
In Water . . . . 5 mm.
After 6 hrs. boiling . . . 1 mm.

Tensile Test—neat.

7 days.	*28 days.*
600 lb. sq. in.	695 lb. sq. in.
600	605
580	655
610	640
640	685
655	670
6)3685	6)3950
mean 614 lb. sq. in.	658 lb. sq. in.

Tensile Test, 3 to 1 Sand Cement.

7 days.	*28 days.*
415 lb. sq. in.	480 lb. sq. in.
405	410
405	510
415	540
385	520
460	580
6)2485	6)3040
mean 414 lb. sq. in.	507 lb. sq. in.

Density of Sand.

The weight of 1 litre of sand was 1,580, 1,589, 1,584, 1,585, 1,586 and 1,587 grams in each of six tests; an average of 1,587 grams, equivalent to 99 lb. a cu. ft.

TABLE II.

Weight of Concrete in lbs. per cu. ft.

% Cement.	Weight when made. (Calculated.)	After 24 hours in air.	After 28 days in water.
100	137	134	137·5
90	138	134·4	138
80	140·5	136	140
70	142	138·5	142
60	144·3	141	144
50	146	143·5	146
40	147·5	146	148
30	150	147	149
20	150	148	150
10	151·6	149	150

TABLE 3.

Crushing Stresses in lbs. per sq. inch in 28 Days Water.

% Cement.	Max.	Min.	Mean.	From curve.	Safe working stress to give factor of safety = 4.
100	8,770	8,500	8,635	8,635	2,160
90	8,030	6,890	7,460	7,790	1,948
80	7,370	7,330	7,350	7,000	1,750
70	6,820	5,340	6,080	6,260	1,565
60	6,120	5,310	5,715	5,830	1,458
50	5,870	5,850	5,860	5,650	1,412
40	5,820	5,170	5,495	5,495	1,374
30	5,150	5,000	5,075	5,075	1,268
20	4,300	3,500	3,900	3,980	995
10	2,440	2,350	2,395	2,350	587

EFFECTS OF CEMENT CONTENTS ON CONCRETE.

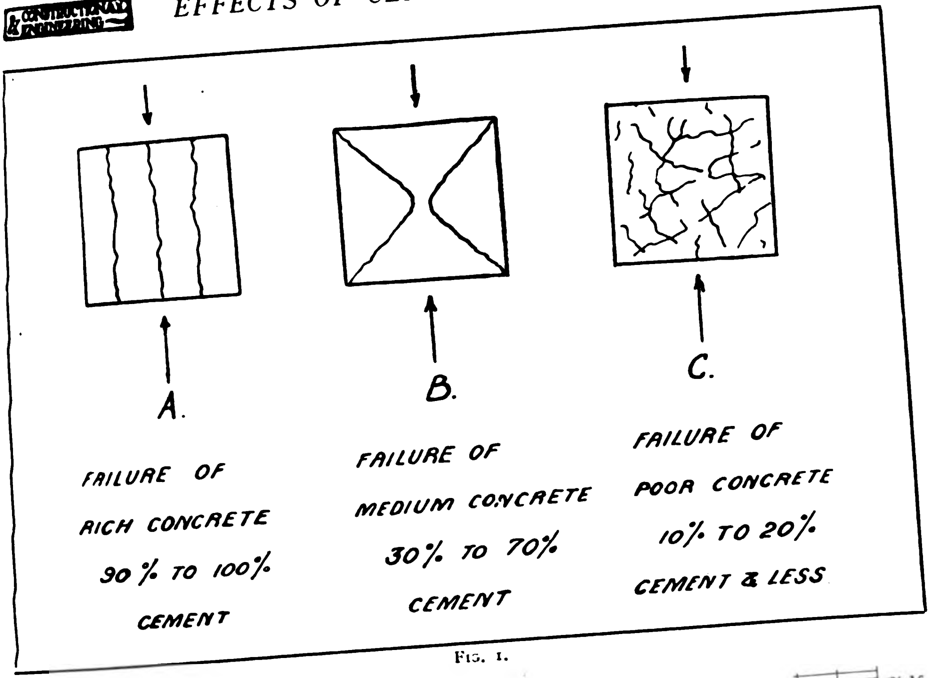


Fig. 1.

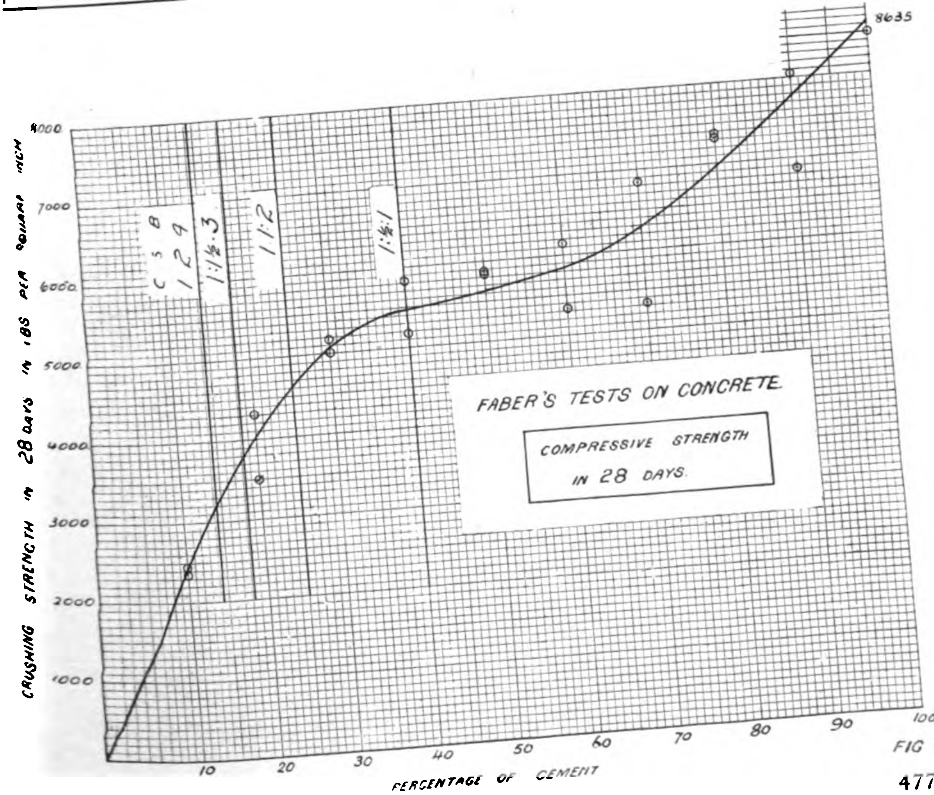


FIG 2

Density of Ballast.

The weight of 1 litre of ballast was 1,387, 1,389, 1,389, 1,394, 1,393, 1,399 grams. in each of six tests ; an average of 1,392 grams, equivalent to 87 lb. a cu. ft.

CONDUCT OF TESTS.

The proportions of cement, sand, and ballast were calculated so as to give 5,000 gms. of dry materials in all cases, the cement contents being 10 per cent., 20 per cent., 30 per cent., etc., up to 100 per cent. in the various tests, the remainder being divided between the sand and ballast in the proportions of 1 to 2.

These materials (*see Table* 1) were weighed out, mixed, and the water calculated by the formula given was measured out and added.

Two cast-iron 4 in. cube moulds were filled with concrete, and the remainder of the concrete carefully kept.

After keeping for 24 hours in air, the cubes and residue were weighed, and the cubes then kept in water for 28 days, when they were crushed.

The temperature during mixing did not vary outside the limits of 58° to 61° Fahr., and all the materials had been kept in the laboratory to acquire its temperature. The specimens were also kept approximately at this temperature when hardening in water.

CRUSHING.

The cubes were crushed in an "Amsler" machine. In most cases timber packing was used. The tests marked with an asterisk gave low results owing to a knot in the timber. The average in the curves has taken cognizance of this in giving these tests less importance. The five poorer specimens were crushed without packing.

It was very marked that the richness of the concrete affected not only the strength but the method of failure. The richest specimens (100 per cent. and 90 per cent. cement) failed by splitting vertically. The medium ones (70 per cent. and 30

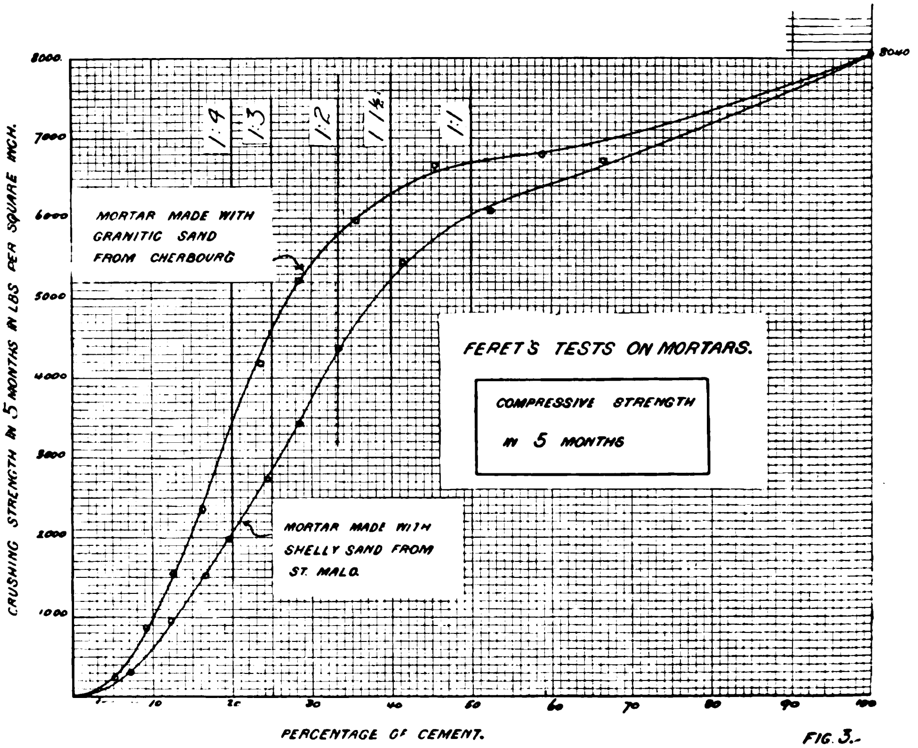


FIG. 3.-

per cent. cement) gave the usual conical failure, while the poor ones (10 per cent. and 20 per cent. cement and less) failed by irregular crushing. These are illustrated in *Fig.* 1.

Fig. 2 shows the relationship between the percentage of cement and the crushing strength of the cubes. This curve is one of the greatest importance and brings out many interesting points.

It will be noticed that the positions on the curve corresponding to 1 : 2 : 4 and 1 : 1 : 2 mortars are indicated so that the relative strength of these and other similar proportions can readily be ascertained.

The crushing strength of 1 : 2 : 4 concrete from the curve is 3,150 lb. per sq. in. and that of the 1 : 1 : 2 concrete is 4,600 lb. per sq. in. It will be noticed that this is almost exactly 50 per cent. greater than the 1 : 2 : 4. This is a matter of considerable interest and has some bearing on the L.C.C. Regulations, which allow 600 lb. per sq. in. for 1 : 2 : 4 concrete and 750 lb. per sq. in. for 1 : 1 : 2 concrete. In other words, they only allow a 25 per cent. increase in stress against the 50 per cent. increase in stress actually ascertained by the experiments.

Fig. 2 also brings out the interesting fact that between 40 per cent. of cement and 70 per cent. of cement there is only a very small increase in strength. After that, however, the increase in strength with additional cement contents again becomes appreciable. This is a very interesting fact and can readily be understood if the strength of the concrete is considered in terms of its tensile strength across vertical or oblique planes. It is of interest to note that the curve between compressive strength and cement contents has quite a different shape for concrete from its shape with cement mortars.

Fig. 3 gives the corresponding curves drawn from results published by R. Feret in 1897. It will be seen that the curvature in the case of the mortar curves is quite different, and a comparison of the two is of considerable interest.

Perhaps the next interesting fact emerging from these experiments is in connection with the shrinkage, that is to

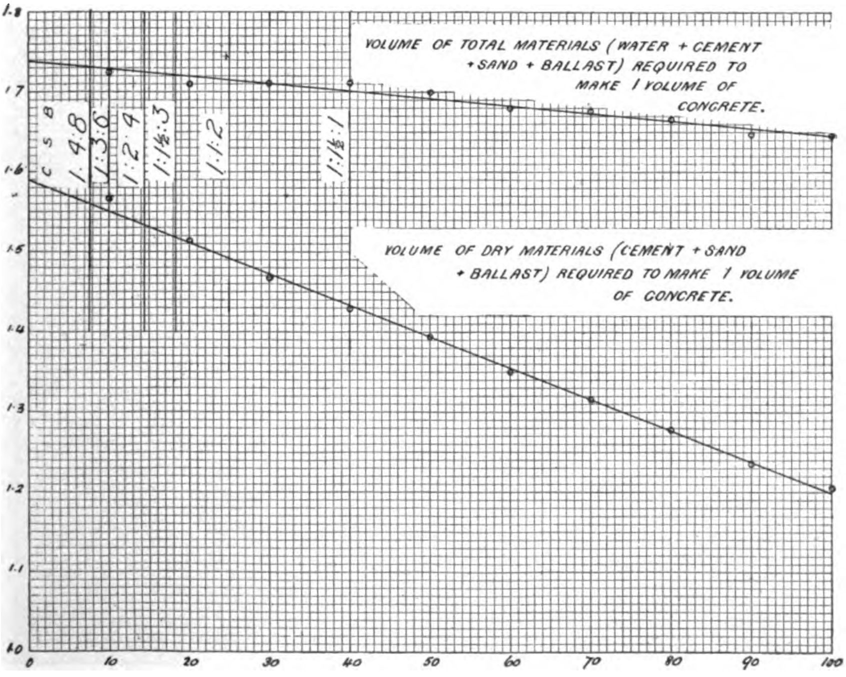


FIG. 4.

say, the reduction in volume of the dry materials which takes place when they are mixed together and wetted to form concrete. To ascertain this, not only were the materials actually weighed as mentioned and the volume of the concrete in the cubes ascertained, but the residue of concrete left over when the cubes had been made was also carefully measured so that the total quantity of concrete produced from a given quantity of material was definitely ascertained. This shrinkage is, of course, of enormous importance to contractors, because from it can be calculated how much cement and materials are required to produce a cubic yard of concrete.

In the last column of *Table* 1 the shrinkage is expressed as a percentage of the volume of dry materials, and it will be seen that it rises consistently with the richness of the mixture from about 18 for neat cement to about 36 for a 10 per cent. mixture. In other words, 100 parts of dry material were required to produce 64 parts of concrete, or that 100 ÷ 64 = 1·56 cu. yds. of dry material are required to produce 1 cu. yd. of concrete of this particular richness. Of this 1·56 cu. yds. the cement would represent, roughly speaking, 0·16 cu. yds., leaving about 1·4 cu. yds. to represent volume of sand plus volume of ballast measured separately.

Generally speaking, the results of these tests confirm those of many other previous tests, which all go to show that the amount of shrinkage is generally greatly underestimated by contractors when estimating for concrete.

In the calculation of these shrinkages, cement was taken as weighing 90 lb. per cu. ft. In practice it may actually weigh less, depending on its condition, and in such cases the shrinkage would actually be greater. *Figs.* 4 and 5 show the relationship between the volumes of cement, sand, ballast, and water, and the volume of dry materials and total materials, and should be of considerable

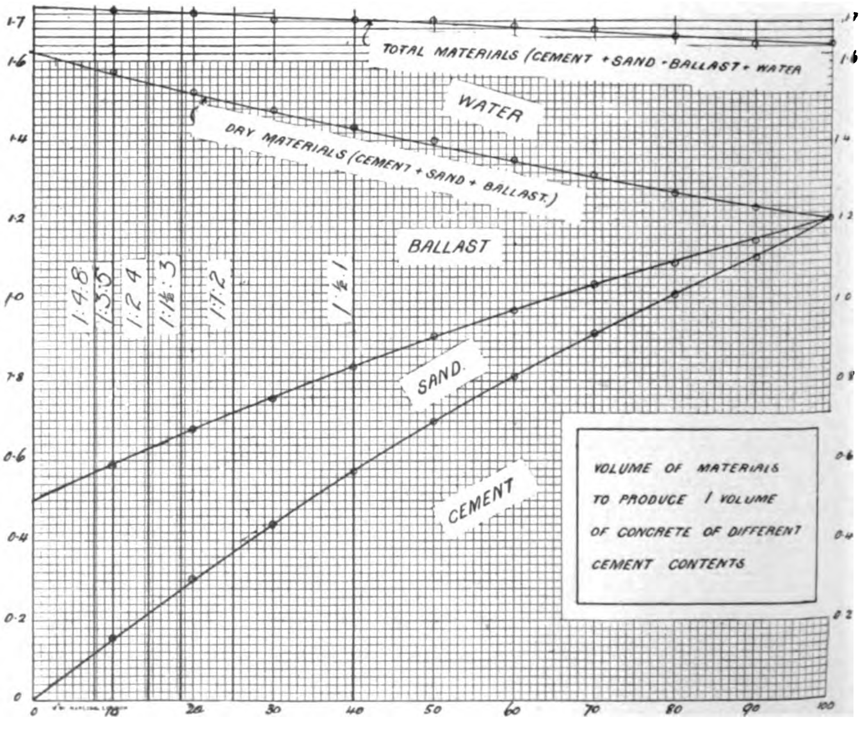


FIG. 5

assistance to contractors in estimating. The necessary figures for these curves are given on *Table* 1a, which were calculated from the figures in *Table* 1.

Table 2 gives the weight of concrete in lb. per cu. ft. for the different mixtures at different periods, from which it will be seen that the weight per cu. ft. increases as the cement content is reduced. These results are also shown on *Fig.* 6. They also show that the cubes lost about 3 per cent. of water by evaporation in the first 24 hours and re-absorbed about the same amount in the subsequent 28 days under water.

Table 3 gives the crushing stresses in lb. per sq. in. at 28 days in water. In column 5 from the left is given the value taken from the curve in *Fig.* 2 so as to work to a uniform curve and avoid the little irregularities of experiments. The last column gives the same value divided by 4 so as to represent the stresses which should be justified in practice.

If we interpolate between these values, we find that the safe stress for a 1 : 2 : 4 concrete works out to 3,150 ÷ 4, which is nearly 787 lb. per sq. in. This is, of course, considerably higher than the present L.C.C. allowance of 600, and it would appear that the figure of 750 lb. per sq. in. which the lecturer had for some time been urging the L.C.C. to adopt in the revised Regulations, is fully justified by the experiments.

As a matter of interest in passing, the corresponding safe stress to give a factor of safety of 4 for the 1 : 1 : 2 concrete comes to 4,600 ÷ 4 = 1,150 lb. per sq. in. Naturally, he would only use these stresses when the materials were equal to those used in the tests.

In the experiments just described, the lecturer expresses his indebtedness to Mr. H. T. Desch and Mr. Derek Hannen, who co-operated with him in conducting the tests and provided the necessary facilities, and whose interest aided materially in securing such accurate and consistent results.

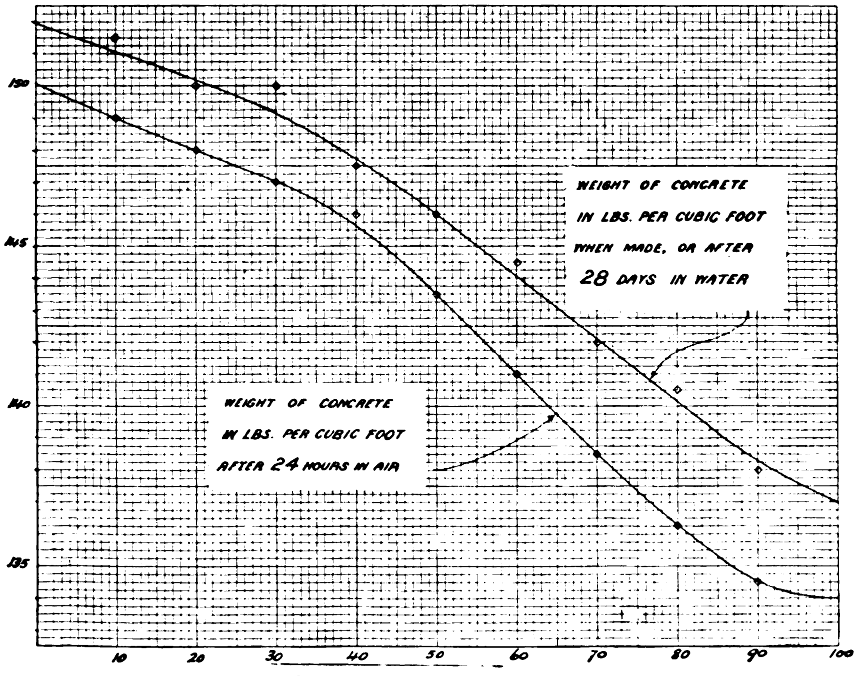
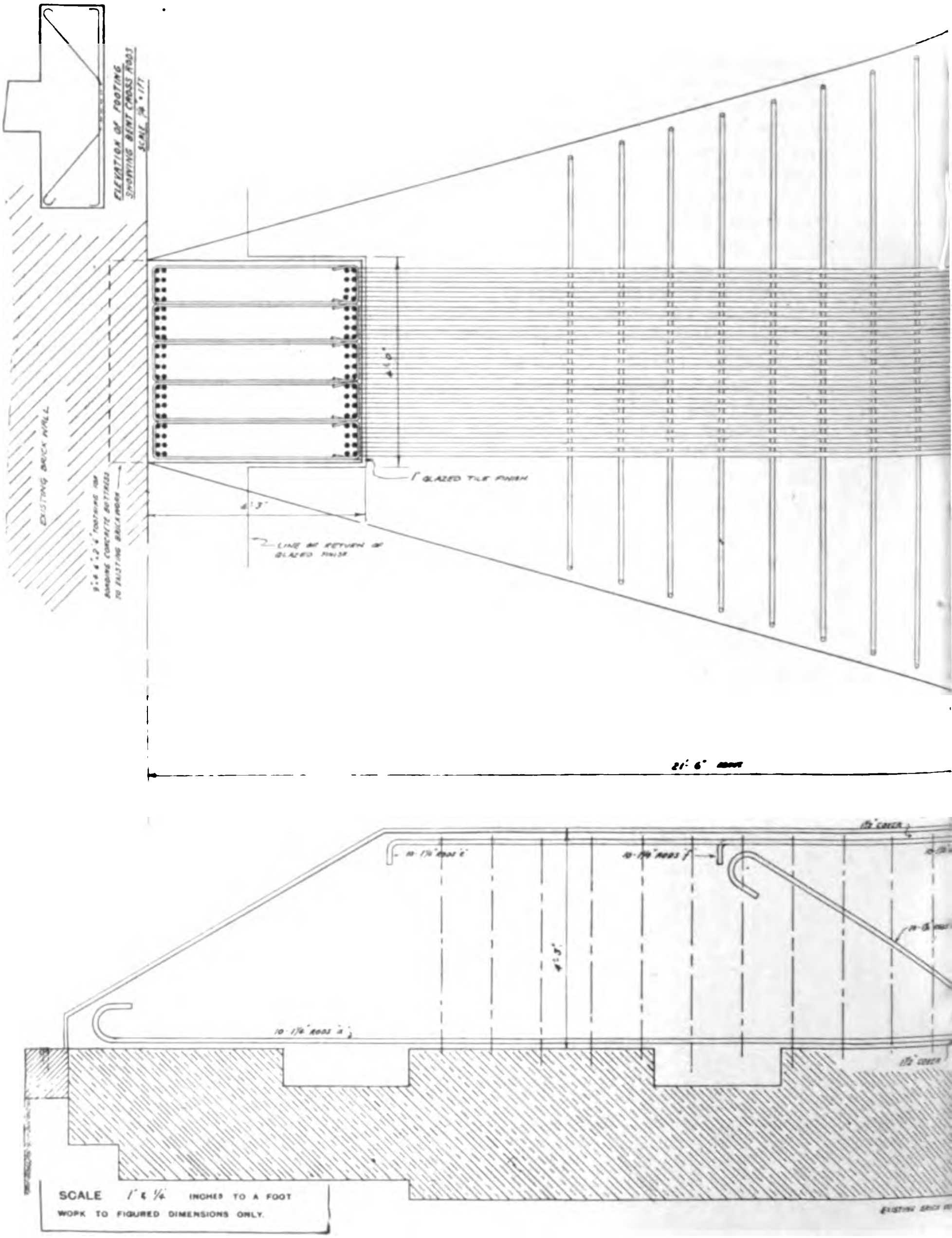


FIG 6


ELEVATION OF FOOTING
SHOWING BENT CROSS RODS
SCALE 1/8" = 1'
EXISTING BRICK WALL
9'-6" × 2'-6" FOOTING FOR
BONDING CONCRETE BUTTRESS
TO EXISTING BRICKWORK
4'-0"
1" GLAZED TILE FINISH
4'-3"
LINE OF RETURN OF
GLAZED FINISH
21'-6"
1½" COVER
10-1¼" RODS 'E'
10-1¼" RODS 'F'
4'-3"
10-1¼" RODS 'A'
1½" COVER
EXISTING BRICK
SCALE 1" & ¼ INCHES TO A FOOT
WORK TO FIGURED DIMENSIONS ONLY.

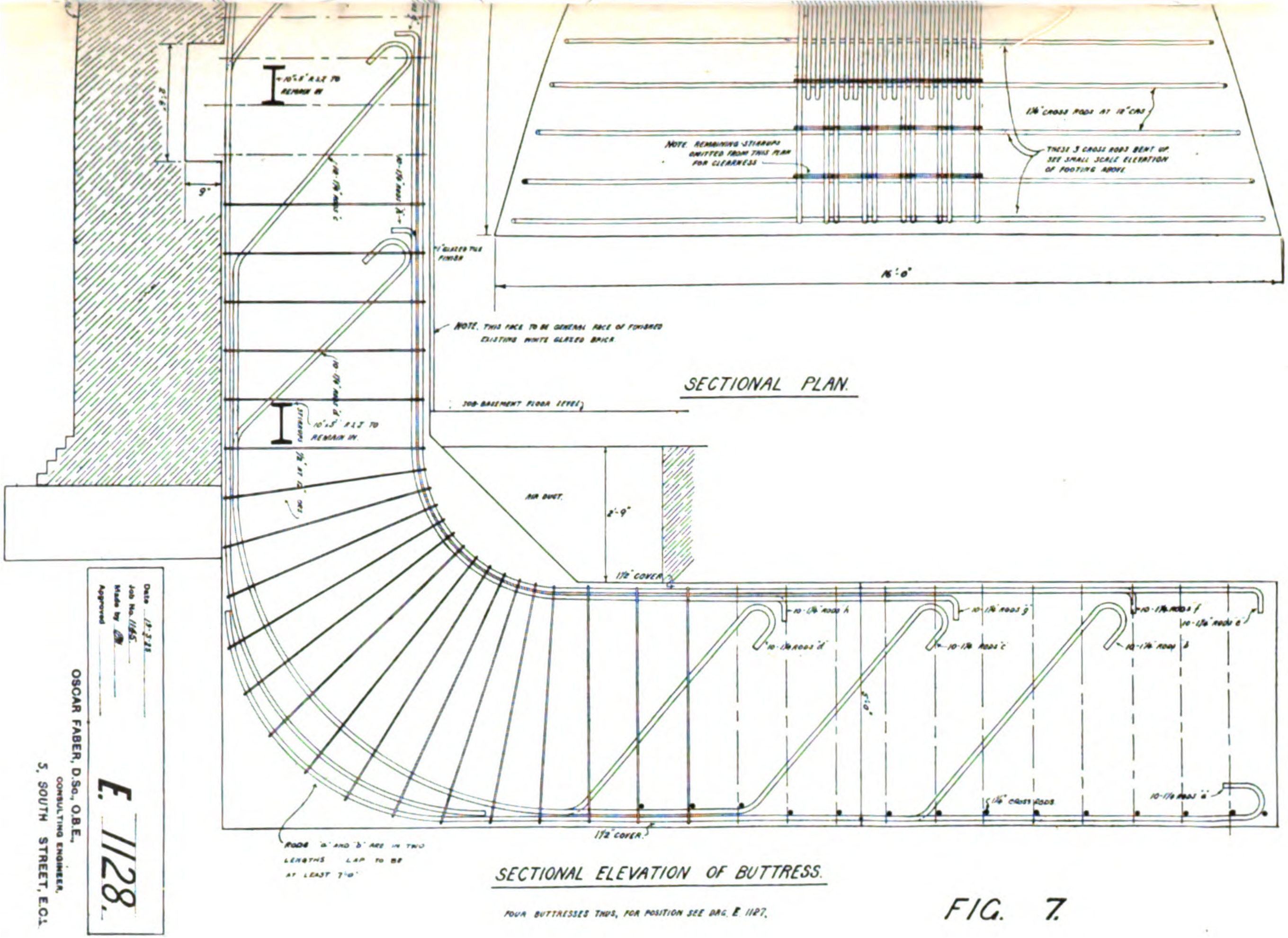

1¼" CROSS RODS AT 12" CRS
THESE 3 CROSS RODS BENT UP. SEE SMALL SCALE ELEVATION OF FOOTING ABOVE
NOTE. REMAINING STIRRUPS OMITTED FROM THIS PLAN FOR CLEARNESS
16'-0"
SECTIONAL PLAN.
NOTE. THIS FACE TO BE GENERAL FACE OF FINISHED EXISTING WHITE GLAZED BRICK.
1" GLAZED TILE FINISH
SUB-BASEMENT FLOOR LEVEL
10"×5" R.S.J. TO REMAIN IN
10-1¼" RODS 'c'
10-1¼" RODS 'X'
10-1¼" RODS 'd'
10"×5" R.S.J. TO REMAIN IN.
STIRRUPS ½" AT 12" CRS
2'-6"
9"
AIR DUCT.
2'-9"
1½" COVER
10-1¼" RODS h
10-1¼" RODS g
10-1¼" RODS f
10-1¼" RODS e
10-1¼" RODS 'd'
10-1¼" RODS 'c'
10-1¼" RODS b
5'-0"
1¼" CROSS RODS
10-1¼" RODS 'a'
1½" COVER
RODS 'a' AND 'b' ARE IN TWO LENGTHS LAP TO BE AT LEAST 7'-0"
SECTIONAL ELEVATION OF BUTTRESS.
FOUR BUTTRESSES THUS, FOR POSITION SEE DRG. E 1127.
Date 19-3-25
Job No. 1165
Made by
Approved
E. 1128.
OSCAR FABER, D.Sc., O.B.E.,
CONSULTING ENGINEER,
5, SOUTH STREET, E.C.1.

FIG. 7.

EFFECTS OF CEMENT CONTENTS ON CONCRETE. CONCRETE

PART II.

EXPERIMENTS ON THE VARIATIONS OF VOLUME IN PROPORTIONING CONCRETE.

Engineers, architects, and contractors have been in the habit of specifying concrete to be apportioned by volume. For example, a common specification is 4 parts ballast, 2 parts sand, and 1 part of cement, all measured by volume. To this custom there are grave objections owing to the fact that a cu. ft. of cement may represent anything between 60 and 90 lb., depending on the extent to which it is consolidated or to which, on the other hand, air has been occluded between the cement particles, and in the same way, but to a much lesser extent, the amount of sand in a given volume varies with its wetness and its degree of consolidation.

He was aware of this discrepancy, but it was Mr. Desch who first opened his eyes to the extent of the variations; which he had never realised were so great as they are, and it was with a view to ascertaining this in a different manner that experiments were put in hand to show how much a given volume of cement and sand could vary in weight under different conditions. The actual cement used in the previous experiments was found to weigh 1,432 grams per litre when put in the litre pot and shaken until it was thoroughly consolidated. As 1,445 grams per litre corresponds to 90 lb. per cu. ft. it will be seen that this was a common figure.

The cement was then filled into the litre pot by allowing it to trickle in from a vessel held some distance above it through a very small orifice, so that the cement dropped in a thin stream through the air and settled in the litre pot without any shaking. When it had filled the pot and some distance above it, the excess over the top of the pot was cut off with a trowel and the contents were weighed. The weights in two separate experiments were 955 grams and 950 grams respectively per litre. This corresponds to about 60 lb. per cu. ft. and represents only two-thirds of the previous weight. For this particular cement the two figures of 90 lb. per cu. ft. and 60 lb. per cu. ft. represented the maximum and minimum weights of the cement which could be obtained by variations in compaction obtained by shaking.

It was thought to be interesting to see what actual weights would be obtained by scooping the cement with an ordinary cement scoop direct from the barrel and filling the litre pot without any shaking. The results of four consecutive experiments were as follows:—

1,105 grams per litre.
1,206 ,,
1,074 ,,
1,079 ,,

These results lie intermediate between the maximum and minimum values previously recorded, and lie rather below than above the mean, and correspond roughly to about 70 lb. per cu. ft.

The significance of these variations is, of course, as follows:—The proportions by volume for concretes are generally based on cement weighing 90 lb. per cu. ft., but actually the cement may weigh as little as 60 lb. after it has been sufficiently disturbed and is poured into the cement measure as loosely as possible. In such cases the engineer thinks he is getting 6 to 1 mixture on the basis of 90 lb. per cu. ft., when actually he would only be getting a 9 to 1 mixture on that basis, and in practice would actually get something which would lie intermediate between the two values.

When the cement is loaded into barrels or sacks at the works its weight per cu. ft. approximates to 70 and its transit by rail and carrier has the effect of consolidating it so that it frequently weighs about 90 lb. per cu. ft. when it arrives at its destination. What it would weigh in the actual measure on the banker board would depend entirely on the way in which it is filled into such measure, whether it is shaken or filled loosely. It is clearly most unsatisfactory that variations of this magnitude up to 33 per cent. in the cement contents can occur, and this is equally unsatisfactory from the engineer's or architect's points of view as well as that of the conscientious contractor, and to obviate this difficulty the lecturer considers it very much better to specify concrete as follows:—

.. cu. ft. of ballast.
.. cu. ft. of sand.
1 bag of cement containing not less than 200 lb.

Experiments were made to see to what extent the same variations occurred in the case of sand and ballast. It was realised, of course, that they would be much less, but it was desirable nevertheless to ascertain their actual values.

PART III.

The lecturer was recently called in to advise in connection with some retaining walls. A large site had been excavated to a depth of 30 ft. approximately, and the surrounding earth, together with the buildings upon it, were being retained by means of a brick wall 3 ft. in thickness. At the time when he was called in the wall was showing signs of overturning owing to the pressure of earth behind it. It was immediately shored to render it safe and subsequently strengthened as described.

In connection with this work the lecturer formed the opinion, after an examination of the soil, that the pressure on the back of the wall might safely be calculated at a pressure of $p = 30 (h + 6)$ where "p" is the horizontal pressure in lb. per sq. ft. and "h" is the depth in ft. from the top of the soil. The effect of the buildings behind the wall could be allowed for by an equivalent surcharge of 6 ft. of earth.

On this basis, the horizontal pressure at the top of the wall would amount to 180 lb. per sq. ft., and that at a depth of 30 ft. to 2,080 lb. per sq. ft. It was desired, for several reasons, to have a check of these pressures if possible, and it was found that the trench for the retaining wall was strutted across by means of horizontal timber struts 7 ft. apart in both directions, the sectional area of the struts varying between 7 in. sq. and 8 in. sq.

This, of course, was on the portions of the wall where the dumpling had not been removed, and where the struts were still resisting the full horizontal pressure corresponding to their particular depth. It was noticed that the struts had indented themselves to a considerable distance into the horizontal timber walings, and this suggested the means of checking the earth pressure. A particular strut half-way down the wall was found to be 7½ in. sq., and the indention to the horizontal timber which amounted to about ⅜ in. was measured. A similar strut and a similar piece of waling were then tested in a testing machine and it was found that a pressure of about 28,000 lb. was required to produce the same indention of the strut into the waling.

Having regard to the fact that these struts were 7 ft. apart in both directions it was possible to check back and calculate that the earth pressure at this depth must be of the order of $28{,}000 \div 49 = 570$ lb. per sq. ft. The calculated pressure at that depth by his assumed pressure curve comes to 630 lb. per sq. ft., which was considered to be in close agreement.

A further check on this happened to be possible. It appeared that near one corner of the job the dumpling was being removed and the timbering was being supported by soldiers, which in their turn rested against a horizontal steel joist, 18 by 7 in section on a 34 ft. span, resting at its two ends against vertical stanchions in the wall, which were themselves temporarily strutted with timber. The arrangement of timber was such that the joist carried the full pressure from a depth of 7 ft. 6 in. below ground to a depth of 22 ft. 6 in. below ground. On the lecturer's assumed pressures, this corresponded to a total distributed load of 945 lb. per ft. run of joist, or 32,250 lb. as the total load distributed uniformly on the joist, approximately equal to 14½ tons.

The calculated deflection of this on a span of 34 ft. amounted to .93 in., and it was found by measurement that the actual deflection of the steel joist was 1 in. as accurately as it could be measured, which, again, is in close agreement with the calculated figure based on the earth pressure given above. This was considered of sufficient interest to be recorded, because it is not often engineers have opportunities of checking the pressures for which retaining walls need to be designed.

As a matter of interest, *Fig.* 7 (pp. 482–3) is added showing how the walls were actually strengthened. A reinforced concrete buttress of rather peculiar design was constructed against the inside of the wall at intervals and efficiently bolted to the existing brick walls. Its effect is to increase the leverage of the weight of the wall from about 1 ft. to about 15 ft., and is therefore obviously extremely effective in supporting the wall against any tendency to overturn. It will readily be appreciated how extremely economical this method of strengthening the wall was as compared, for example, with the construction of a continuous reinforced concrete wall inside the brick one, which would have been the only practicable alternative which would have given adequate strength to resist the pressures which actually were found to exist.

From a point of view of obstruction, the arrangement adopted also proved to be peculiarly convenient, as these buttresses could be placed in certain recesses in the brick wall. It will be noticed that the buttresses are very heavily reinforced, as might be expected, as the pressure on a considerable width of wall was being resisted by a buttress only 4 ft. wide, and it was necessary to reduce the dimensions to an absolute minimum with a view to causing as little obstruction as possible.

DISCUSSION.

Mr. **Ewart S. Andrews** said in structural engineering in many respects some of the fundamental information upon which engineers must base their practical design was often lacking. The lecturer had given definite information on an essential point, namely, the actual increase of strength to be obtained by increasing the richness of the mixture and by keeping the conditions as far as possible constant in other directions. The importance of the water content could not be too highly stressed; it seemed useless to spend a great deal of trouble and ingenuity in carefully working out the necessary reinforcement when designing if the mixing were done in an indiscriminate manner. On the question of specifying the mixture, the lecturer had recommended the form which the speaker thought to be the only practical solution of the difficulty, namely, specifying a certain weight of cement and volumes of other materials.

Mr. H. K. G. **Bamber**, dealing with the question of using cement by weight rather than by volume, said he agreed with the lecturer, but it must not be forgotten that 1 cwt. of fine cement contained many more particles than did 1 cwt. of coarse cement, so that it was important for the engineer to obtain the finest possible cement on the market. The lecturer's experiments had been made with one particular aggregate, namely, sand and gravel, but the calculations depended entirely on the kind of aggregate used. The aggregate used by the lecturer was impervious to water, and as a result he had been able to proportion the water and obtain certain definite tensile and crushing strengths. It was to be hoped the paper would be the precursor of others dealing with different kinds of aggregates, many of which would freely absorb water. It was impossible for a contractor using an aggregate with a very large percentage of water to dry it in order to arrive at the percentage of water to be used, and engineers should not accept altogether those definite proportions of water as applicable to other aggregates. As to the hardening qualities of cement, he did not think it was quite realised to what extent modern cements had improved. Portland cements of to-day were immensely superior to anything stipulated in the British Standard Specification, and it did not seem to be realised how very much more valuable for constructive purposes were the very finely-ground cements of to-day compared with the coarse-ground cements of the past. The tensile and crushing strengths of cement to-day mixed with 3 parts of sand was as strong at 24 or 48 hours as the British Standard Specification required at 7 days.

Mr. H. **Kempton Dyson**, referring to the control of wetness, said he thought the slump test ineffective, and, as regards actual work on the job, he believed that the ordinary means of measuring an adequate wetness were good enough; but undoubtedly the quantity of water varied a good deal in practice, and he did not think it practicable to gauge it very accurately. He believed the most practicable way to secure good concrete on a job was to have rather an excess of water, so that the concrete would work into place easily. He did not favour a 1 : 2 : 4 concrete, because he thought the sand and cement were not in sufficient volume to make sure of filling the interstices between the coarse material. He had found in practice that a better concrete was obtained by making a proportion of, say, 1 cement, 2½ sand, and 3½ coarse material. It was very often found with a 1 : 2 : 4 mixture that when the centering was removed there was a honeycomb effect, because it had not worked down properly; therefore, more sand should be used than laboratory experiments suggested as a basis. The laboratory experiments referred to in the paper were very interesting, but he would not be willing to accept the figures given by the lecturer as being a proper gauge of the working stresses to be put upon concrete of various proportions in practice. He thought ordinary 1 : 2 : 4 concrete stressed to 600 lb. per sq. in. was stressed as highly as it should be, and he would not care to reckon upon a factor of safety of 4 from laboratory experiments. The great danger in practice was that contractors did not take proper care of cement, but stored it in a shed through which damp air blew, and in a week or a fortnight it might reduce the strength of the cement by 25 or even 50 per cent. With regard to the gauging of cement, he believed the measuring of cement in a gauge-box had been disfavoured for many years. The Concrete Institute in its Standard Specification for Reinforced Concrete made a particular point that the cement was to be proportioned by weight, on the basis of 90 lb. being the weight of a cu. ft. of cement. Compacted cement did weigh about 90 lb. per cu. ft., but it could be made to weigh almost anything, according to the way in which it was tipped out. He always saw that the gauge-boxes were made to suit the sacks of cement obtained, and he always had two separate gauge-boxes, one for sand and the other for coarse material, the sand-box being the smaller. If a flat, shallow box were put on the banker, filled with stones and levelled off, and then a small box stood on the top and filled with sand, the lower box could be withdrawn, and the coarse materials would run out on the banker. If the sand-box were then lifted off the sand would flow over the stones. The bag of cement could then be emptied on the top, and the cement smoothed over the whole heap, so that part of the mixing was already effected.

Mr. G. A. **Gardner** said if the tests were made for the purpose of establishing a new standard for regulations he should have thought flexural tests would have been the better tests to make, on the ground that the stresses were more complicated and that they would be a more direct measure of the stresses allowed in practice. He agreed with the lecturer with regard to raising the allowable working stress; if the lecturer's figures were taken, and the factor of safety of 4 applied to them, no doubt the real factor of safety in actual practice would be somewhere about 3, which would appear ample if the work were properly designed.

Mr. H. T. **Desch**, speaking with regard to fineness of cement, said it would be an advantage in settling the proportion of cement to sand and ballast to take into consideration the apparent density of the cement, as determined by means of the standard apparatus introduced some years ago by the French and the Germans. He had used cement reckoned at 80 lb. per cu. ft., 78 lb. per cu. ft., and even 72 lb. per cu. ft., producing excellent concrete for the purpose intended. In his opinion cement should always be proportioned by weight and never by volume, as seemed to be generally done. As to mixing, he did not remember to have seen concrete used to the best advantage where cement was measured in boxes. As regarded the amount of water to be used, unless one were dealing with reinforced concrete, where there was a large amount of steel in proportion to the concrete, he maintained that the dryer one could use the concrete the better would be the result. As an example of the money that could be saved by carefully proportioning the cement in concrete, according to the quality of the cement, it would be found that with the best quality of cement now obtainable a mixture of 1 : 2½ : 5 could be used for ordinary work, instead of the usual 1 : 2 : 4, reckoning the cement at 90 lb. per cu. ft. in each case.

Mr. S. **Bylander** said he did not agree that it was not practicable to measure the quantity of water used in a mix; on several very large jobs he had made sure that the water had been measured. He was an advocate of machine mixing, and in a machine mixer there was no difficulty in fixing apparatus for measuring the water. He agreed with proportioning the cement by weight, and not by volume. In Canada it was usual to have bags of cement weighing 87 lb., which were easy to handle, and with machine mixing it was practicable to use one bag of cement for each mix and proportion the other aggregates in accordance.

Mr. **Marshall** pointed out the difficulty in practice of keeping to the percentage of water, because, according to whether the day was damp or sunny, the water content had to be altered. Very often the sand and the aggregate were wet.

The **President** (Mr. E. Fiander Etchells) referred to experiments he had carried out with cement paste, the results of which showed close agreement with those of the lecturer. Dealing with water content, he said different kinds of cement at different ages required different percentages of water, and it was possible, by the ball test as well as the slump test, to ascertain the amount of water to be added. If a ball with a definite water content were dropped from a distance of about 20 in., the cement would alter in shape from the spherical to the shape of an orange; with 1 per cent. less of water, the ball would break at its edges, but with 1 per cent. more it simply flattened out.

Dr. **Faber**, replying to the discussion, pointed out that he had not said that the amount of water was always to be measured in practice. He had found it necessary for the purpose of his experiments, in order to get consistent results, to measure the water, and had merely found a formula which gave constant wetness throughout the tests. In reply to Mr. Bamber, he said it was only possible to work with one definite aggregate in one series of tests, and he had confined himself to one and had done the work thoroughly. He agreed that there was a very great improvement in modern cements, and he would have thought it might be reasonable to draw the conclusion that, with the same mixture of concrete, using a very much better cement, a correspondingly higher stress was justified. What was the use of improving cement year after year out of all recognition and still using the same stress per sq. in. with exactly the same proportion of cement as ten years ago? There was then no advantage in having a better cement. Under the British Standard Specification cement had to be very very bad if it were to fail; nearly all cements actually used gave strengths at least 60 per cent. in excess of the Specification, and he appealed for a Standard Specification which represented modern requirements and conditions. He believed that any concrete to which one could properly apply the word "sloppy" was not a concrete which should be recommended for reinforced concrete. As to whether it was better to use more sand and less coarse material—a 1 : 2½ : 3½ mixture instead of a 1 : 2 : 4—that would depend on the nature of the aggregate, i.e., according to whether it consisted of rounded pebbles or sharp angular pieces of broken stone, such as granite, which had much greater voids. It should also depend on the purpose for which the concrete was to be used, whether for a reservoir or a retaining wall for retaining relatively dry materials not subject particularly to excess of corrosion. As to the criticism of laboratory experiments for settling stresses to be used, tests on concrete cubes could not be carried out except in a laboratory. His tests had given a factor of safety of 4 in 28 days, which showed the increased stresses he had suggested; the L.C.C. regulations specified a factor of safety of 3 in 28 days. At any rate, the tests gave a sufficient margin between what could be obtained with a properly-made concrete and what should be obtained in practice. If in practice they could not get three-quarters of what was obtained with these cubes, then it was a poor practice, and should not be encouraged. As to using broken brick, that had a great many serious disadvantages, at least in reinforced concrete work. Concrete made with broken brick had very great shrinkage, and therefore tended to produce rather objectionable contraction cracks in floor slabs, for example, which were not produced to nearly such an extent, if at all, if impervious materials, such as ballast, were used. He did not feel much sympathy with the objection to measuring cement in the way he had suggested on the score that there might be in existence a different size of sack. Nine sacks out of ten in this country contained nominally 204 lb. of cement, and one would never receive a sack of any other size unless it were specially asked for. In reply to Mr. Gardner, he did not agree that flexural tests would have been better for the purpose in view, nor did he quite understand Mr. Gardner's reasoning that they would have been better because the stresses were more complicated. It seemed to him that that fact rather went against the suggestion. With regard to higher permissible stresses, the proper course was to specify stresses which would be safe in ordinary competent hands. Dealing with the ball and the slump test, he said his experience of the former was that it only applied to a concrete which was drier than any concrete which it would be proper to use for reinforced concrete work, and his experience of the slump test was that it was useless. He would far sooner control water by the eye, unhampered by a slump test, than with the slump apparatus in use.

A NEW SAND WASHER.

A PATENT sand-washing machine, recently put on the market by Messrs. Glenfield & Kennedy, Ltd., of Kilmarnock, embodies some interesting new features. The washer, which is contained in a tank 6 ft. by 4 ft., consists essentially of a series of small washing chambers, one behind the other on an inclined plane, connected together underneath by a cast-iron feeder-pipe forming the bottom of the plane. The dirty sand is thrown into a large main hopper in front of the apparatus, and falls through into a small receiving chamber underneath, which is inclined upwards. This chamber is at the bottom end, and is fitted with a brass nozzle through which a jet of clean water is blown at a pressure which need not exceed 5 lb. This water-jet blows the dirty sand upwards and out of the inclined container box as it falls continually from the hopper, and passes it in a stream into an oscillating strainer-box suspended by springs and operated by a small hydraulic motor. This action separates all stones and foreign material, and the clean sand falls through the strainer into a chamber underneath, at the bottom of which is a second water-nozzle. The intimate mixture of the falling sand and the water-spray under pressure results in the great bulk of the impurities being washed out immediately and carried over a weir in the settling chamber in a stream of dirty exit water, whilst the clean sand falls by gravity continuously to the bottom of the chamber. From here it is then blown by another water jet, up the inclined feed-pipe at the bottom of the apparatus, where it enters the first of a series of small washing-chambers. These contain at the bottom a smaller water-nozzle which carries the sand upwards for a certain short period, the clean water rushing through the sand and carrying the impurities, which are lighter than the sand, upwards and out of the top of the chamber in a stream of dirty water, whilst the heavier sand is only carried upwards sufficiently to fall into the next washing chamber higher up on the inclined pipe, where the process is repeated, there being three such chambers in series. In this way the sand is propelled continuously through the entire apparatus by a series of water-jets, and the action consists in repeatedly washing the sand with successive jets of clean water and thus separating the impurities, whilst the action is aided considerably by the scrubbing action of each grain of sand on its neighbour, because of the repeated changes of direction of the travel in the machine. The clean sand finally issues out of the end of the machine in a continuous stream. It is claimed that with only 5 lb. pressure of water at the nozzles 4·15 tons of sand can be washed per hour. The machine is specially designed to work at low water-pressure and to obviate the difficulties of having to obtain a high-pressure supply, but in the latter event the duty of the machine is correspondingly increased.

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

***Readers are cordially invited to send in questions relating to concrete, which will, as far as possible, be answered direct and subsequently published where they are of general interest. We cannot give complete designs; these are the province of the consulting engineer. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.*—Ed.**

FLOOR LOADS.

Question.—A floor slab, 4 ft. 9 in. by 12 ft. centre to centre of supports, is carried on reinforced concrete beams. The floor is worked out to carry a super load of 10 cwts. per sq. ft. The reinforcement the short way consists of ½ in. round bars at 6 in. centres and the long way ½ in. round bars at 9 in. centres. The bars the short way are continuous and bend up over the supports. The total thickness of the slab is 5½ in. I should be glad if you would inform me what would be the safe concentrated load on this slab at a point exactly in the centre of the bay—that is, a load such as would be got from the wheel of a heavily-loaded vehicle—and also what would be the safe rolling-load from such a wheel. The beams may be taken as being amply strong, but I should be glad if you would tell me a method of working out the safe concentrated load for the slab, assuming that this concentrated load be the only load on the slab.—J. W. D.

Answer.—The moment at midspan due to a super load of 10 cwts. a sq. ft. plus dead load would be approximately

$$(1120 \times 4\tfrac{3}{4}) \times \frac{57''}{12} = 25{,}000 \text{ in. lb.}$$

The percentage of steel is

$$\frac{\cdot 4 \times 100}{4\frac{3}{4} \times 12} = \cdot 7 \text{ per cent.}$$

Hence the resistance moment would be about

$$R = 95\, bd^2 = 95 \times 12 \times (4\tfrac{3}{4})^2 \cdot$$
$$= 25{,}000 \text{ in. lb.}$$

and we agree that the slab is able to carry a super load of 10 cwt. Strictly speaking, there should be top steel in the adjacent panels to justify $\frac{Wl}{12}$ with such heavy live loading.

The moment due to a concentrated load at midspan would be

$$M = \frac{WL}{4}$$

neglecting continuity. It is unwise to allow much for continuity with heavy rolling loads on small spans unless there is top reinforcement in the adjacent panels, which, we take it, there is not. Considering this moment spread over a width of slab equal to the small span, the moment per ft. would be

$$M = \frac{WL}{4} \times \frac{1}{L} \text{ ft. lb.}$$
$$= \frac{W}{4} \text{ ft. lb.}$$
$$= 3W \text{ in. lb.}$$

Hence, $3W = 25{,}000$ in. lb.

$$W = \frac{25{,}000}{3} = 8{,}300 \text{ lb.}$$

approximately.

This would only apply when there is ample cross reinforcement, as in the present case.

To allow for impact, it would be wise to consider only about 4,150 lb. as the safe rolling load.

⁂ Since receiving our reply to his question, our correspondent writes : "It may interest you to know that a floor slab constructed as described in my letter was tested by running a Foden steam wagon over it, the B.A.W. of which was 8 tons (that is, 4 tons per wheel). The deflection in the centre of the slab—measured in 64ths of an inch—was 0·4, and the long beams supporting the slab 0·2, making the net deflection of the slab 0·2 of a 64th = $\frac{1}{320}$ in. As the permissible deflection is $\frac{1}{800}$ times 4 ft. 9 in. = approximately $\frac{1}{16}$ in., this seems to show that the floor slab is amply strong enough for its purpose."

CONCRETE OIL TANKS.

In the course of a tentative recommended standard practice for construction of concrete oil storage tanks, prepared by a special committee of the American Concrete Institute, it is pointed out that in such structures care should be taken in the design to provide for all external and internal stresses; namely, hydrostatic pressure of contents, external hydrostatic pressure from the ground on the floor and walls, earth pressure on walls, and live and dead loads on roof.

In continuous or restrained members, positive and negative bending moments should be given equal consideration. Temperature reinforcement should be placed in the walls and floor independent of other reinforcement, the steel ratio being not less than one-third of one per cent. The maximum range in temperature should be predetermined as a basis of calculation.

While the hydrostatic pressure of water in soils is about 50 per cent. of the full hydrostatic pressure, it is recommended that not less than 52½ lb. per sq. ft. for the full head of water be assumed and the difference allowed as a factor of safety for unforeseen conditions.

Steel in tension, whether in circumferential tension or in tension due to negative bending moments on the face of tank walls or floor exposed to the oil, should be designed for a safe working stress of 10,000 lb. per sq. in. In circumferential walls, the thickness of the concrete should be based upon a maximum tensile strength in the concrete of 150 lb. per sq. in., but a minimum thickness of 8 in. at the top and 10 in. at the bottom of walls is recommended to give space for spading.

All tanks should be built with a concrete roof. The roof should be made gas-tight at the junction with walls, and should be constructed without unprotected openings.

At least two manholes should be provided for each tank 20 ft. in diameter and over, and one manhole for tanks of less diameter. The multiplicity of manholes is recommended so that oil vapours or gases may be more quickly diffused when the tank is opened prior to being cleaned out or other work being done within.

The tanks should be set on firm, well tamped earth, rock or other suitable foundation to guard against settlement. The foundations should be constructed to provide full bearing for the tank bottom.

REINFORCEMENT.

The reinforcing metal should meet the requirements of the current standard specifications for billet steel reinforcement except that twisted square bars should not be employed in the construction. Reinforcement should be free from excessive rust, scale, paint or coatings of any character which would tend to reduce or destroy the bond.

Wherever it is necessary to splice the reinforcement no lap-splice should be less than forty diameters. No two laps of adjacent rods should be directly opposite each other in circular walls.

Reinforcing steel should be cleaned of all mill and rust scales before being placed in the forms. All reinforcement should be bent or curved true to templates, placed in its proper position and securely wired or fastened in place well in advance of the concreting. Reinforcement should be inspected and approved by the engineer before any concrete is deposited.

CONCRETE SPECIFICATIONS.

The cement should meet the requirements of the current standard specifications for Portland cement.

Fine aggregate should consist of natural sand or screenings from clean, hard, tough crushed rock or pebbles graded from fine to coarse particles passing, when dry, a sieve having four meshes per lin. in.

Coarse aggregate should consist of clean, hard, tough crushed rock or pebbles graded in size, and should contain no soft, flat or elongated particles. The size of the coarse aggregate should range from 1 in. down, not more than 5 per cent. passing a screen having four meshes per lin. in. and no intermediate sizes should be removed.

Crusher-run stone, bank-run gravel or mixtures of fine and coarse aggregates prepared before delivery on the work should not be used because the ratio of fine to coarse material varies so widely as to lead to concrete mixtures of greatly varying proportions.

The concrete should be mixed in a proportion by volume not leaner than 1 cu. ft. of Portland cement, 1½ cu. ft. of fine aggregate and 3 cu. ft. of coarse aggregate.

All concrete should be mixed by machine in a batch mixer with charging hopper, water storage, and a water measuring device which can be locked.

The quantity of water used in mixing should be the least that will produce a plastic or workable mixture which can be worked into the forms and around the reinforcement. Under no circumstances should the consistency of the concrete be such as to permit a separation of the coarse aggregate from the mortar in handling. An excess of water should not be permitted as it seriously affects the strength of the concrete, and any batch containing such an excess should be rejected.

CONSTRUCTION.

Where continuous placing of concrete in floor and walls is impracticable, the operations should be in the following order: (1) footings and floor; (2) walls; (3) columns, if any; and (4) roof.

No break in time of over forty-five minutes should occur during any one operation except between columns and supported roof slabs, where six hours should elapse to permit the settlement of concrete in the columns. In placing concrete in floors, it should not be allowed to set up on exposed vertical faces where work is temporarily discontinued. Column footings should be placed monolithically with the floor, and the floor reinforcement so designed as to distribute the column load over a sufficient area.

In walls the concrete should be placed in layers or not over 12 in. for the entire wall, so that a monolithic structure will result.

When deposited the concrete should be thoroughly spaded against the inner and outer faces of the forms so that it will densely compact and force out the trapped air and work back the coarser particles from the face of the forms. More and better work can be accomplished by using light wooden sticks 1 in. by 2 in., planed smooth, with a sheet steel blade at lower end, than with heavy spades.

The floor and roof should be levelled with a straight-edge or strike-board, finished with a wood float, and trowelled to a smooth surface as soon as possible after the concrete is deposited. Voids in walls, if any, should be filled with a 1 : 1½ mortar as soon as the forms are removed. After filling these void spots they should be rubbed smooth with a fine-grained carborundum brick.

The forms should not be removed until the concrete has sufficiently hardened so that no deflection or damage will result. In warm weather column and wall forms should remain undisturbed for at least 48 hours and roof forms at least 7 days. In cold weather no predetermined rules can be made, but should be left to the judgment of the engineer.

Unless the roof is insulated against temperature changes by sufficient earth cover, or the reinforcing in walls and roof is designed to take care of temperature stresses likely to occur, an expansion joint should be provided between the tops of walls and the bottom of the roof slabs so that any expansion of the roof due to temperature changes will not transmit bending moment into the walls. In roof slabs where temporary stops are necessary they should be made on the plane of least shear; that is at the middle of beams or slabs.

If the walls and floor are not deposited in one operation an approved joint or dam should be provided between the floor and walls. It can be made as follows: (1) provide a recess in the floor to engage the wall and insert a galvanised iron strip at least 10 in. wide with joints riveted and soldered so as to form a continuous band on one side of the recess, or (2) place a 10-in. strip of deformed sheet metal 1 in. back from the inside form and engaging floor and wall equally. After the wall form is removed the 1-in. recess is to be plastered with a 1 : 1½ mortar to make a 6-in. coved base.

The interior of the tank should be oil-proofed by persons familiar with this process.

The tank should be tested as soon as practicable by filling with water, and should show no signs of leakage during a period of 7 days. The tank should not be placed in service until the concrete has gained sufficient strength to resist all involved stresses.

Backfilling should not be done around the walls or deposited on the roof until after the water test has been made and at such a time when it can be safely done.

All steel reinforcement should be interconnected and grounded by an approved method as a protection against lightning. Any pipe passing through the tank should be properly bonded to the reinforcement.

An independent, permanently open, galvanised-iron vent-pipe terminating outside the building should be provided for every tank. The lower end of the vent-pipe should not extend through the top into the tank for a distance of more than 1 in. Vent openings should be screened with 40 by 40 non-corrodible wire mesh or its equivalent (preferably cone shaped), and should be of sufficient area to permit the inflow of liquid during the filling operation and in no case less than 1½ in. in diameter. Vent-pipes should be provided with weatherproof hoods, and terminate 12 ft. above the top of the fill pipe, or, if tight connection is made in filling line, to a point 1 ft. above the level of the top of the highest reservoir from which the tanks may be filled, and preferably not less than 3 ft., measured horizontally and vertically from any window or other building opening.

The end of the filling pipe in the tank should be turned up so as to form a trap or seal, and should be as remote as possible from any door or other opening so as to prevent liability of flow of oil through building openings. The terminal should be outside the building in a tight, incombustible box or casting, so designed as to make access difficult by unauthorised persons.

All pipes should pass through the roof, if possible. If necessary to pass through the walls, there should be flanged sections with a space of about 1½ in. left between the flange and the concrete on each side of the wall; this space should be caulked later with litharge and glycerin or other approved oilproof material. It is advisable also to have a ring projecting about 2 in. around the pipe-sleeve which engages the concrete.

Concrete Signposts in Gloucestershire.

The Gloucestershire County Council is now using concrete direction posts on the main roads in the county. The posts, arms, and finials are cast separately and assembled on the site.

Concrete and Iron Roads.

A new form of road construction is being experimented with by the Manchester Corporation on a portion of Pall Mall, Market Street. A stretch of this street which sustains a fairly heavy traffic is being laid with concrete blocks upon which is a surface of cast iron. The concrete blocks, which are about 6 in. square and 3 in. deep, have a ½ in. plate of cast iron keyed into them.

The Institution of Structural Engineers.

Under the auspices of its Western Counties Branch, who were responsible for the arrangements, a large number of members of the Institution of Structural Engineers recently paid a visit to Gloucester. A civic reception was accorded to the party by the Mayor of Gloucester in the afternoon, and afterwards visits were paid to Gloucester Cathedral, an exhibition of artificial stone, and the Exhibition of Gloucester Industries. A meeting in the evening was addressed by the President (Mr. Etchells), the President-Elect (Major Petrie), and the Chairman of the Western Counties Branch (Mr. Pimm).

Relative Costs of Concrete and Brick Cottages.

The Local Government Board of Ireland has written to the Carrick-on-Suir Urban District Council pointing out that the tender accepted by the Council for the erection of six houses in brick worked out at £114 per house more than a tender submitted which provided for the houses to be built of concrete blocks. The Board requested the Council to reconsider its decision, and to draw up a specification for the erection of the houses with concrete block walling.

CORRESPONDENCE.

OVERHEAD MOTOR ROADS.

SIR,—It is apparent, in the face of present conditions, that something will have to be done, and soon, to relieve the traffic congestion on the principal streets of London, which are even now too congested to permit of efficient and rapid transit to and from the railway termini, docks, warehouses, industrial houses, etc. One scheme is obviously out of the question, and that is to construct an underground road, not from the engineering standpoint but from the point of view of finance, for as shown by previous undertakings the approximate cost of a subway 50 ft. wide would be £1,000,000 per mile. Also the laying down of new roads in central London is equally out of the question, due to the high number and cost of existing buildings that would have to be demolished to provide sufficient space for the new highway.

In the first place, I may say that practically every railroad entering the Metropolis can produce an example of "elevated railroad" constructed as viaducts to span over water, roads, buildings, and factories, and my contention is this —if railway, why not roadway? Taking, for instance, an elevated roadway stretching from the London Docks or Aldgate to Marble Arch, and also one from the Surrey Docks to Clapham Junction, it can be clearly seen that the road would have to be at least 80 ft. in height to obviate the possible obscuring of light and to clear the average building height.

I firmly believe reinforced concrete to be the best material for this work, due to its very low cost of upkeep, for, although very injurious to steelwork and masonry, the atmospheres of London have little or no effect on reinforced concrete structures. As regards design, I have in mind certain footbridges, one over the River Seine in Paris and the other over the Great Central Railway at Sheffield. These bridges are constructed of steel and concrete respectively, and are built on arch and cantilever principle, and it is my suggestion that the roadway should be carried on a viaduct constructed on the same lines, only the arches, in our case, being continuous. Also I consider that reference should be made to the patent which does away with the use of centering in reinforced concrete bridge-work.

As an alternative I would suggest a steel bridge built on the same lines as say the Manhattan Valley Viaduct of the Rapid Transport Co., of New York, U.S.A., the principle being known as the "parabolic braced arch system." This type of viaduct is obviously more attractive than the ordinary box or warren girder or the lattice braced girder similar to the railway bridges at Charing Cross (S.E.C. section of the Southern Railway) and at Blackfriars, which are acknowledged eyesores over the Thames.

Everyone is aware that the idea of overhead motor roads is openly ridiculed and scoffed at, but taking everything into consideration I honestly think that the overhead road is the only solution to a problem which is rapidly coming to a climax, the reaching of which would mean practically death to small transport concerns which exist only by reason of the low cost of working only attainable by rapid transit and efficiency which are well-nigh impossible at the present time.

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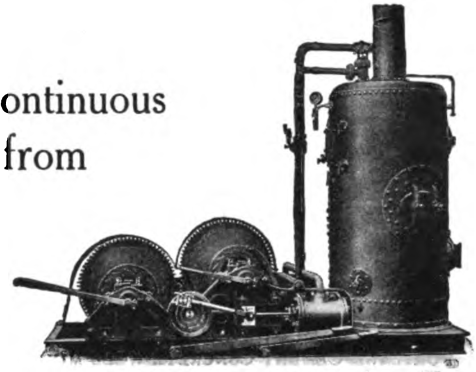

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Do. do. to sides and soffits of beams average 9 in. by 12 in.	„ „	1	0
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Labour, splay on ditto	„ „	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

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MEMORANDA.

PROSPECTIVE NEW CONCRETE WORK.

Aberystwyth.—*Railway.*—The G.W.R. directorate have authorised, at an estimated cost of £100,000, the reconstruction of Aberystwyth railway station. The plans provide for extension of platforms and covering, improved and rearranged station buildings, doubling of Llanbadarn line (length 1 mile 4 chains) and extension of locomotive depot.

Adwick-le-Street.—*Sewage.*—The U.D.C. has approved a sewage disposal scheme for Carcroft Ward, the cost of which is estimated at £49,577.

Birmingham.—*Railway Station.*—At the last meeting of the Corporation it was stated that the L.M. & S. Railway proposes to provide a passenger station between Adderley Park Station and Stechford Station.

Boston.—*Road.*—The U.D.C. is considering a scheme for the construction of a new road in the parish of Leverton, at a cost of £2,085.

Bournemouth.—*Groynes.*—The T.C. has received the sanction of the Ministry of Health to a loan of £48,500 for the construction of a sea-wall and promenades and reinforced concrete groynes.

Bradford.—*Houses.*—The Ministry of Health has sanctioned a loan of £46,200 for the erection of 132 houses, also a loan of £17,480 for 66 tenement dwellings at Longlands.

Caerleon.—*Roads.*—The Council is considering the construction of two new main roads, one from the " Ship " Hotel, crossing the river and joining the Usk Road, and the other through the racecourse to the Newport Tramway terminus.

Camberley.—*Tennis Courts.*—The U.D.C. is considering the provision of six hard tennis courts.

Carnforth.—*Road.*—The U.D.C. is considering a proposal to construct a new road to connect Preston St. and Lancaster Rd.

Eastbourne.—*Groyne.*—The Corporation proposes to reinforce with concrete an existing groyne at Langney Point.

Falmouth.—*Houses.*—The East Kerrier (Penryn) R.D.C. has passed plans for the erection of between 300 and 400 houses on the Bolsowick estate.

Farnham.—*Sewage.*—The Sewage Works Committee is borrowing £2,470 for the extension of the sewage works.

Glasgow.—*Road.*—The Ministry of Transport has intimated to the Glasgow Corporation that subject to the approval of certain details it is prepared to make a grant of 50 per cent. towards the proposed new road to Duntocher, estimated to cost £500,000.

Hull.—*New Streets.*—The Corporation has approved plans for five new streets on land lying between the Beverley Road and the Skidby drain.

Ilfracombe.—*Road.*—The U.D.C. has under consideration a scheme for the construction of a new road from the Harbour to Hillsborough Road.

Leamington.—*Road.*—The T.C. has received permission from the Ministry of Health to borrow £12,788 in connection with the construction of the new road from the Corporation destructor works to Old Warwick road.

Liverpool.—*Roads.*—The L. Corporation is developing a scheme for the construction, at an estimated cost of £3,000,000, of a new arterial road from Liverpool through Oldham, Bury, Bolton, Rochdale, and other industrial centres.

Morecambe.—*Road.*—The Corporation has applied to the Ministry of Transport for a loan of £39,000—half the cost of a proposed ring road.

Newton Heath.—*Roads.*—The Manchester Corporation has received sanction from the Ministry of Health to borrow £28,918 for the construction of roads.

Oswestry.—*Street.*—The T.C. has passed plans for a new street from Weston Lane towards Edward Street.

Port Said East.—*New City.*—The Egyptian Cabinet has considered a proposal of the Suez Canal Co. to plan a new city on the eastern bank of the canal, to be called Port Said East. The company proposes to advance 10,000,000 francs for this purpose.

Portstewart.—*Houses.*—The U.D.C. has approved plans for 25 terrace houses to be erected on a new road.

Settle.—*Sewage Works.*—The U.D.C. require to borrow £4,000 for the improvement of the sewage disposal works.

Wellingborough.—*Road.*—The U.D.C. proposes constructing a new road from Doddington Road to Northampton Road.

TENDERS ACCEPTED.

Burton-on-Trent.—*Bridge.*—The Corporation has accepted the tender of Gray's Ferro-Concrete Company, Glasgow, at £47,000, for the widening of Burton Bridge over the Trent. The work is to be done in reinforced concrete on the Hennebique system.

Canterbury.—*Pipes.*—The T.C. has accepted the tender of Messrs. H. C. Gullar and Co., Kingsway, W.C., at £1,648, for laying about 198 yds. of 36 in. concrete tubes and works for the electricity works.

Dewsbury.—*Sewage.*—The T.C. has accepted the tender of Messrs. Wellerman Bros., at £6,145 5*s.* 6*d.*, for sewage disposal work.

Hendon.—*Bridge.*—The Hendon D.C. and the Middlesex C.C. have accepted the tender of Messrs. Kirk & Randall, at £782, for the construction of a concrete bridge at Shirehall Lane, Hendon.

Hull.—*Sub-station.*—The Corporation has accepted the tender of Messrs. Sangwin, Ltd., Hull, at £782 18*s.* 6*d.*, for a brick and concrete pumping sub-station on the Cottingham Road housing site.

London.—*Raft Foundations.*—The L.C.C. has accepted the tender of Messrs. W. J. Connoll & Co., at £711 3*s.* 9*d.*, for raft foundation for section D of County Hall.

Loughborough (Leics.).—*Bridge.*—The Estates Committee has accepted the tender of Messrs. W. Moss & Sons, Ltd., Loughborough, at £474, for the construction of a reinforced concrete bridge, and the erection of retaining walls and piers in connection with new street entrance at Queen's Park.

Mayo.—*Sewage.*—The Mayo Home and Hospitals Committee has accepted the tender of C. O'Malley, Westport, at £5,450, for the construction of sewage works.

Rastrick.—*Re-building.*—Messrs. Walshaw, Drake & Co. have accepted the tender of Messrs. H. Castle & Co., Brighouse, for a ferro-concrete structure designed by The Standard Concrete Reinforcement Co., Ltd., Bradford.

Shoreham (Sussex).—*Pier.*—The Shoreham Harbour Trustees have accepted the tender of Messrs. G. Munday & Sons, 9 Botolph Lane, E.C.3., at £5,420, for the reconstruction of the existing East Pier at the harbour entrance, in ferro-concrete on the Hennebique system.

RECENT PATENT APPLICATIONS.

187,579.—Buhler Geb : Manufacture of Portland cement.
195,771.—Parker, Winder & Achurch, Ltd., and G. P. Achurch : Concrete mixing machines.
195,859.—M. Kahn : Centering for concrete floors.
195,909.—Ryan Manufacturing and Construction Co., Ltd., and J. Ryan : Manufacturing slabs or blocks.
196,404.—P. J. White (E. Schoepke) : Presses for moulding building blocks, etc.
196,484.—W. G. Bailey : Concrete block-making machines.
196,687.—E. J. Logan : Concrete castings for structural steel and other members.
196,955 and 196,957.—P. L. Lancaster : Construction of concrete piles.
197,005.—T. Shields [Corrugated Bar Co. (Inc.)] ; Reinforced concrete columns.
197,380.—J. Ver Mehr and J. S. Owens : Metallic sheet piling.
197,430.—A. Hiley and E. Le Bas : Concrete piles and columns.
197,735.—F. G. and E. L. Lynde : Apparatus for vibrating concrete.
197,768.—R. Heath and Low Moor, Ltd., and W. E. Harvey : Reinforcing bars.

TRADE NOTICES.

The British Reinforced Concrete Engineering Co., Ltd., of Manchester, have received an order for all the reinforcement required by the Improved Wood Pavement Co., Ltd., in connection with its contract with the Westminster City Council.

A new concrete road is being laid by the Surrey County Council under the direction of Mr. W. P. Robinson, A.M.Inst.C.E., County Surveyor. The road is being laid in alternate bays, and is reinforced for the greater part with Walker-Weston interlocked double-layer reinforcement. The road is in part a widening of the existing road through the parish of Buckland and in part a new road. The width of carriageway is 30 ft. The Walker-Weston Company's reinforcement has also been used by the Plymouth Corporation for reinforcing two streets, Henderson Place and Oxford Place, which were laid as reinforced concrete carriageways. The entire work was carried out under the supervision of Mr. T. Peirson Frank, M.Inst.C.E., until lately Borough Engineer and Surveyor.

The Surrey County Council, under the direction of Mr. W. P. Robinson, Assoc.M.Inst.C.E., County Surveyor, is now laying an all-concrete section of the Surrey East-West Road. The road is reinforced with B.R.C. fabric, made by the British Reinforced Concrete Engineering Co., Ltd., Manchester.

The Dublin Corporation are having carriageways constructed in reinforced concrete on the Donnelly's Orchard Housing Site. B.R.C. fabric is the reinforcement adopted.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

AUGUST 1923 VOL. XVIII. No. 8

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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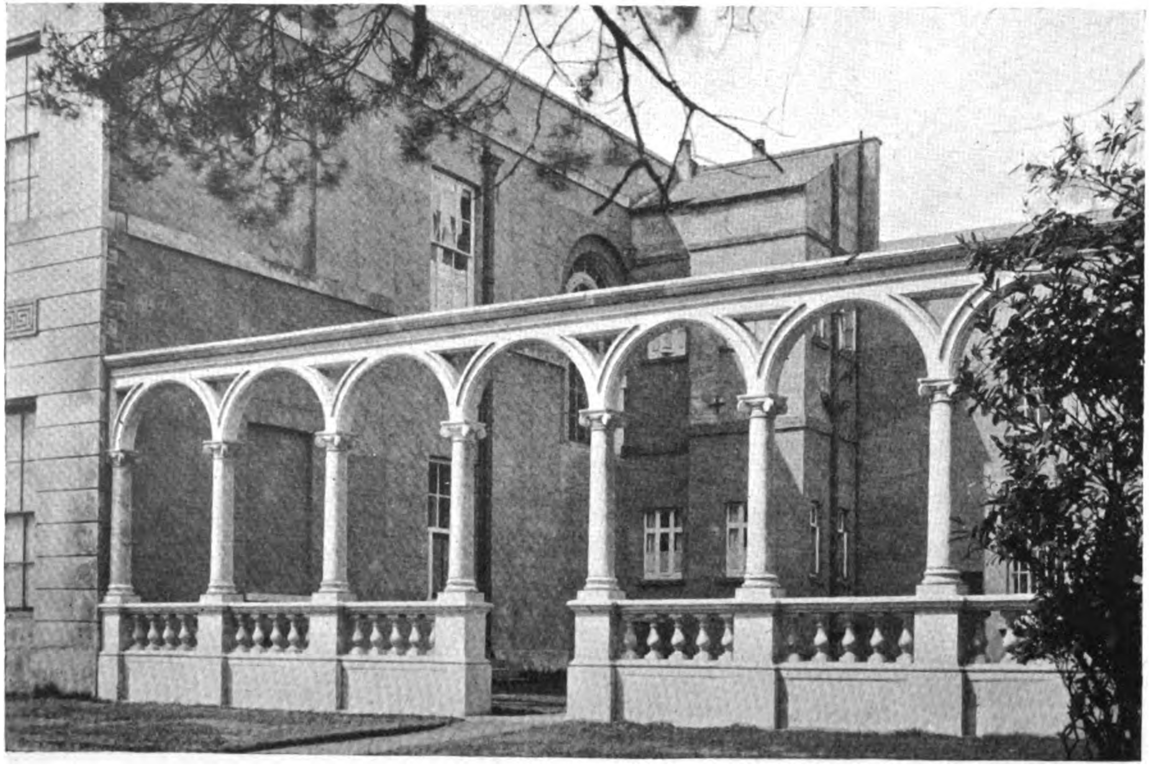

A DECORATIVE SCREEN IN CONCRETE.

[This screen, which is 41 ft. long and 14 ft. high, was built to match the Portland stone of which the existing house is built; and the difference is indistinguishable except on very close examination. The screen was carried out by The Wharf Lane Concrete Co., of Ilminster, Somerset.]

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CONCRETE
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Volume XVIII. No. 8. LONDON, AUGUST, 1923.

EDITORIAL NOTES.

THE BRITISH STANDARD SPECIFICATION FOR PORTLAND CEMENT.

THE paper on the British Standard Specification for Portland Cement recently read by Mr. D. B. Butler before the Institution of Structural Engineers, and given in abstract elsewhere in this issue, embodies the principal suggestions for modification that were made in an article in our issue for July, 1922, and as there was no dissentient voice during the discussion that followed the reading of the paper, we may expect that the new edition of the Standard Specification will contain the more stringent requirements as to strength recommended by Mr. Butler. By this means the standard of quality of concrete in this country will tend to be raised, and the prestige of British cements in other countries will be improved.

In passing, it is curious to observe that a greater prominence seems to be given to testing in the commerce of cement than in any other industry. Sellers, buyers, and users are in accord in putting tests in the forefront of their discussions, and, although this may be partly due to the scientific basis of the manufacture and the sellers' pride in the material, it is probably more the result of the rapid and continuous improvements in quality that have taken place which cause the reputations of five years ago to be eclipsed by present-day performances. Cement testing may have two objects, viz., to compare quality with a fixed standard (or with other cements), and to discover how the cement will behave in practice. It is fairly obvious that the Engineering Standards Committee, which has been responsible for the cement specifications hitherto issued, has laid emphasis upon the former object and regarded cement testing largely as a means of obtaining comparative data. The Specification tests comprise fineness, chemical composition, setting time, soundness and tensile strength, and of these setting time is the only test which has any direct bearing upon the practical use of the cement. In other countries there has been a leaning in the other direction, and in some cases the neat test, which is of purely theoretical value, is not made, while in others compression tests supplement or replace tensile tests, which also are more theoretical than practical in value. There is some ground for the view that the neat and the tensile tests are valuable for diagnosis, and that under the most practical of laboratory conditions compression tests with perfectly clean and perfectly graded aggregates and the minimum of water are still a long way from contract practice. For testing results of practical value users of cement must in fact look to " field tests," as developed in American concrete practice, to supplement the laboratory tests of cement.

The proposed " stiffening up " of the cement Specification leads us to refer to a matter we have previously emphasised, and for which we make no apology in view of its importance. The improvement in the quality of cement is only obtained at extra expense in manufacture, which must be reflected in the price paid by the consumers ; but the consumers fail to benefit so long as they continue to use these improved cements in the same proportions and in concrete of the same bulk as they used the cements of twenty years ago, which were only half as strong.

ALUMINOUS CEMENT.

MR. BUTLER'S paper, referred to above, also referred to a new aluminous cement which possesses the property of rapid hardening, the strength at an age of one or two days being equal to that of Portland cement at twenty-eight days. This cement is composed essentially of aluminate of lime, and in this aspect differs from Portland cement, which is composed for the most part of silicate of lime with only a small proportion of aluminate of lime. The property of hardening rapidly is one of considerable value in certain directions, notably in road repair work, where it is possible to limit the exclusion from traffic to forty-eight hours. For other emergency work this cement will also have advantages, and among other claims it is asserted that piles can be driven three days after moulding.

Concrete users have become so accustomed to waiting for periods of days or even weeks before concrete is sufficiently hard to carry its own unsupported weight or to bear superimposed loads, that it is difficult to envisage the saving in time and economy in procedure that may result if concrete attains in twenty-four hours the hardness usually developed in a month. Subject to certain considerations, it is possible to foresee remarkable developments in connection with aluminous cements. These considerations are endurance and cost.

There has been no published evidence that aluminous cement maintains its high strength and does not deteriorate with time. Concrete blocks made with this cement a dozen years ago and immersed in sea water and water impregnated with sulphate are reported to be intact, but apparently the maximum period at which tensile or compression tests have been made is three years, although again it is reported that there is no evidence of retrogression during this period. There is no reason on chemical grounds to suspect deterioration, but it is obvious that engineers must proceed with caution and the new cement will have to stand the test of time before it is regarded unreservedly as a substitute for Portland cement.

The second consideration is that of cost. Aluminous cements require as one of the raw materials an aluminous mineral such as bauxite or corundum, and neither of these is known to exist in England. Hence at the present time aluminous cement is made only in France and Switzerland, and the price in England is double that of Portland cement. Even if manufacture is commenced in England the aluminous raw material will need to be imported, and this will form a substantial addition to the cost of manufacture, which will also have to carry the additional cost of grinding the very hard-fused material to the great fineness that is necessary. However, with an industry still in its infancy, there are doubtless possibilities of cheapening the manufacturing processes. The high price now current will in fact tend to restrict the use of the cement, and thus the caution which we have suggested is necessary will automatically result. But we do not lose sight of the possibility that in course of time aluminous cements may lead to marked changes in constructional methods, may extend the uses of concrete, and to a certain extent replace Portland cement; at the same time, rendering concrete of still greater importance in the engineering and constructional operations of the world.

THE USE OF COLOUR IN SMALL HOUSES

As announced elsewhere in this issue, the Belfast Corporation has decided to use concrete-block construction for the erection of its new housing scheme, embracing nearly 200 houses, on which work is to start almost immediately. Coming at a time when local authorities and private enterprise are both taking a renewed interest in small house building, in anticipation of the passing into law of the new Housing Bill, it is very gratifying to see that the merits and advantages of concrete are being realised. The schemes built under the previous Housing Acts have now been in existence long enough for definite opinions to be formed on them, and for the position they will occupy in the history of housing reform to be gauged with some measure of accuracy. Although their voice is not so often heard now, critics were not lacking who denounced the new houses no matter of what material they were built—the rooms were too small, the ceilings were too low, the rents were too high, and anyhow they would tumble about the ears of the tenants in a very few years! No doubt such criticism was to have been expected of any large Government programme, but the fact that the schemes were launched by the Government makes it difficult to separate opinions based on careful study from those which were merely political bias. The view of housing reformers and those who look at the subject from an impartial standpoint is, however, that the new houses and cottages mark one of the greatest steps forward of modern times in housing betterment and in improving the conditions under which the working classes live. Whatever criticism may be levelled against them —and it must be remembered that the high costs prevailing at the time had much to do with the smallness of the rooms and the restriction of other amenities which many would have liked to have seen included—the new houses are undoubtedly vastly better than any of the dwellings provided for the poorest of the population in the nineteenth century, and which have become the slums of to-day.

The legacy of slumdom, inherited from the days when the health and convenience of the inhabitants hardly ever entered into the thoughts of a large section of those who provided dwellings for the working classes, is now recognised as one of the chief factors in causing unrest and discontent, and it must be realised that all the efforts now being made to rebuild these areas will be in vain unless the new houses are of a type which will not become the slums of to-morrow. Education in hygiene and civic pride can do much to instil a desire for something better into the minds of the people, but the first essential is that the houses shall be so designed that they will be a pleasure to look at and to live in, and therefore worth looking after. The State-aided schemes already in being prove that such houses can be built at no more cost than the type of dwelling formerly erected for the same class of tenants. It has been proved that porticoes, plastered surfaces which so soon get chipped and unsightly, and other "adornments" which were supposed to attract tenants when they were new, are not at all a necessary adjunct to an attractive dwelling, and that pleasant appearance and economy in first cost can both be secured if attention be concentrated on proportion and materials.

It is undoubtedly the variety in elevational treatment and in lay-out which has made the State-aided schemes so vastly superior to the rows of houses all alike which were previously built. The small house, however, gives but little scope for variety in elevation when a door and four windows are the only features with which it is to be obtained, and it says much for the skill of the architects that monotony is avoided so much as it is; and the bigger the scheme the more difficult it is to give sufficient variety to remove the monotony which any collection of houses all of practically the same size must inevitably possess so far as design goes. The introduction of colour, however, opens up limitless possibilities. With brickwork colour schemes are restricted to shades of red and yellow; relief may be obtained by roughcasting some of the houses in a group, but at a cost which is hardly warranted when the variety it affords is obtained by the use of a material which fulfils no structural function. Concrete, on the other hand, can be coloured to any desired shade at very little cost by adding different oxides during the process of mixing the concrete, thus producing a building material which needs no covering to give it a pleasant appearance. This method was adopted on the large housing scheme at Liverpool, and a deputation from the Belfast Corporation which recently visited Liverpool was so satisfied with the appearance of the colour effect and the way in which it has withstood the weather during the past four years that it has been decided to use the same method for colouring the concrete on the Belfast scheme. The houses thus built not only give variety to large groups of houses as a whole, but each individual house is distinctive, and if this will go any way towards giving each tenant an interest in his house it should have a distinct tendency to prevent the houses developing into slums, and is surely a move in the right direction.

STRUCTURAL STEELWORK REINFORCED WITH CONCRETE.

By **EWART S. ANDREWS, B.Sc., M.Inst.C.E., M.I.Struct.E.**

In recent years structural steelwork and reinforced concrete have progressed upon somewhat parallel lines, and comparatively little has been done in combining the two methods of construction. This is probably due to the fact that the two methods of construction have been regarded as competitors, and the specialists in each of the methods have concentrated upon demonstrating the advantages of the one compared with those of the other.

In actual practice with structural steelwork the steel is very often encased in concrete to increase its fire-resisting properties and to minimise the dangers resulting from rusting. Such encased steelwork is, however, almost invariably designed so as to be capable of carrying the loading without making any allowance whatever for the additional strength of the steelwork due to its casing of concrete.

The aim of this article is to draw attention to the fact that structural steelwork encased in concrete is undoubtedly strengthened thereby, and to give some figures with a view later to deducing rules upon which may be based calculations allowing for this increase in strength.

Before proceeding to a detailed discussion of the experimental data available, it may be pointed out that there are several practical advantages in construction with a comparatively light form of steel frame encased in concrete. The first point to consider is speed of erection; the light structural steel members will be strong enough to support the dead weight of the structure and centering, and so erection can proceed more quickly than in the case of ordinary reinforced concrete because it is not necessary to wait for the concrete to set before proceeding further. Another practical point in favour of reinforced steelwork as compared with ordinary reinforced concrete is that the formwork is simplified and less strutting is required; the formwork is always a matter of some difficulty with contractors for reinforced concrete work, largely on account of the difficulties of forming a sufficiently exact estimate of cost on which to base a price for tender.

The subject will first be considered with regard to columns and then with regard to beams.

COLUMNS.

A number of published results of experiments upon the strength of steel columns encased in concrete are available, and some of them will be considered in detail.

PROFESSOR WARREN'S EXPERIMENTS.[*] —Professor Warren tested some steel columns built up of four 3 in. by 3 in. by ⅜ in. angles with 2½ in. by ⅜ in. lattice bracing. Tests were made of the steel columns alone, and of the columns filled and encased with concrete; in one set of the encased columns the finished size was 12½ in. by 12½ in. just encasing the steel, and in another set a fireproofing cover was employed giving a finished size of 14 in. by 14 in. The concrete mix was 1 : 2 : 3½, and tests were made at 2½ months; the area of steel was 8·44 sq. in. The length of the column was 10·6 ft.

Test pieces of the steel gave a yield point of 39,250 lb. per sq. in., and an ultimate strength of 65,750 lb. per sq. in.

The following results were obtained:—

	lb.
Ultimate strength of plain steel column	= 363,000
Ultimate strength of encased steel column (12½ in. by 12½ in.)	= 678,000
Ultimate strength of encased steel column (14 in. by 14 in.)	= 765,000

Thus the encased columns gave strengths approximately 1·9 and 2·1 times as great as that of the steel column alone.

PROFESSOR BURR'S EXPERIMENTS.[†]—Somewhat similar tests were made by Professor Burr upon two series of columns. One set was composed of four 1¾ in. by 1¾ in. by $\frac{5}{16}$ in. angles, with single diagonal bracing composed of 1¼ in. by $\frac{5}{16}$ in. ties; the other set was composed

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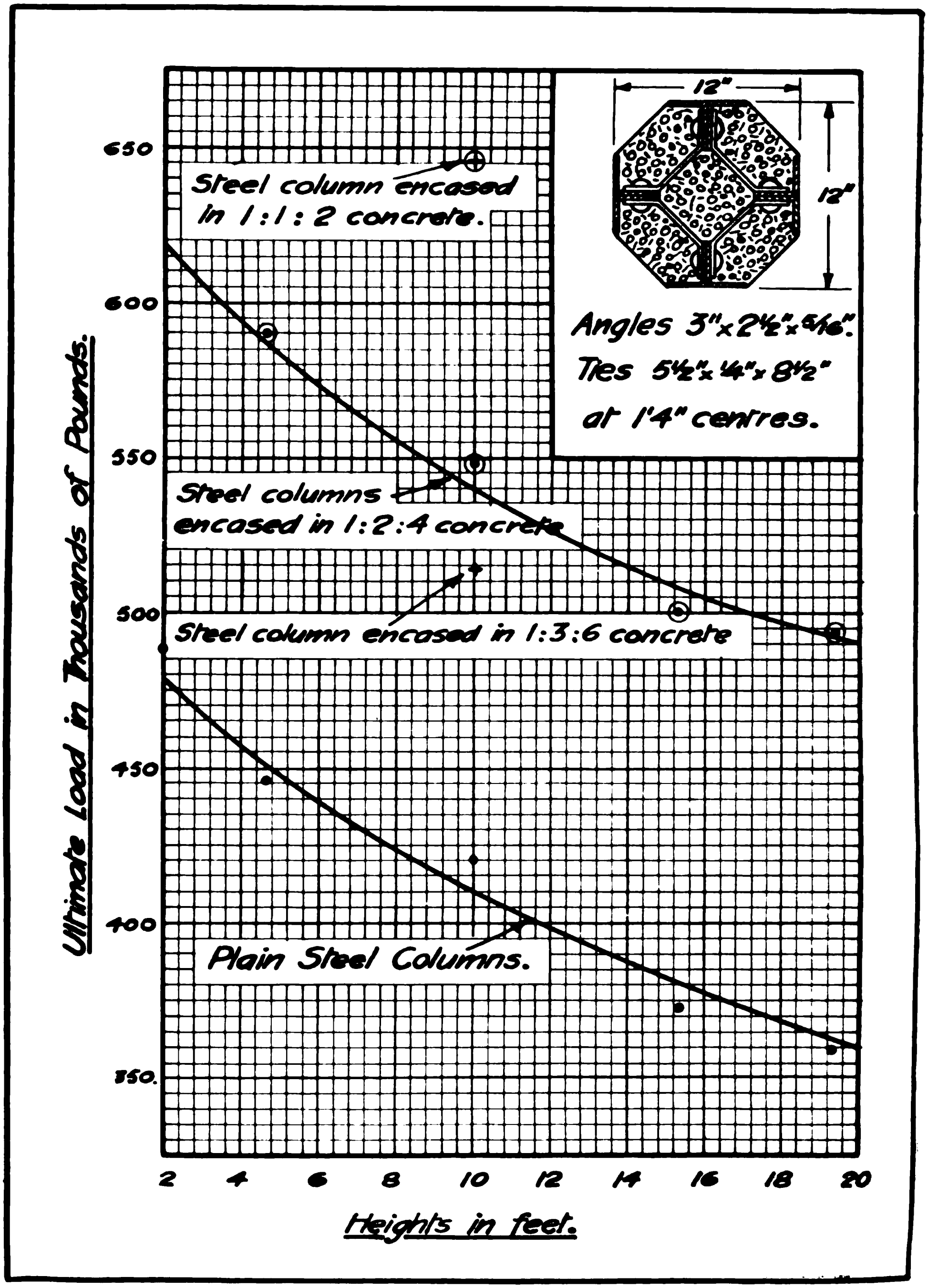
[*] *Engineering Construction.*—Warren, 1921 (Longmans).

[†] *Proc.* Inst. C.E. 1911–12, Part II.


Ultimate Load in Thousands of Pounds.
650
600
550
500
450
400
350.
2
4
6
8
10
12
14
16
18
20
Heights in feet.
Steel column encased in 1:1:2 concrete.
Steel columns encased in 1:2:4 concrete
Steel column encased in 1:3:6 concrete
Plain Steel Columns.
12"
12"
Angles 3"×2½"×5/16".
Ties 5½"×¼"×8½" at 1'4" centres.

of four 3 in. by 1·41 in. channels with 4 in. by ¼ in. transverse battens at 16 in. centres.

The following results were obtained :.

	Lattice (lb.)	*Batten* (lb.)
Ultimate strength of plain columns	= 140,000	155,000
Ultimate strength of filled columns	= 220,000	230,000

The concrete mix was 1 : 2 : 4, and tests were made at 3 months ; the area of the steel section in the case of the lattice set was 4·00 sq. in., and in the case of the batten set 4·76 sq. in. ; the length of column was 7 ft. in each case.

In the case of the lattice-braced columns the filled columns were 1·6 times as strong as the plain columns, the corresponding figure for the other set being 1·5.

ILLINOIS UNIVERSITY EXPERIMENTS.*—A more exhaustive investigation of the problem is described in a bulletin by Professor A. W. Talbot and Mr. Arthur R. Lloyd, of the University of Illinois.

The column section adopted is that known in the United States as the "Gray" column, and consists of four pairs of angles back to back with batten plates between them at 16-in. centres ; the section is indicated upon the accompanying diagram (p. 506).

This diagram shows the strengths of plain steel columns of various lengths, and also the strength at 60 days of the columns of the same sections and lengths encased with concrete in the manner indicated in the section ; points are also shown for the strength of columns 10 ft. long enclosed with 1 : 3 : 6 concrete and with 1 : 1 : 2 concrete.

Tests were also made upon columns of the same type with a spiral reinforcement outside the column and with a wooden casing to give a fireproofed column, but the results of these tests are not given upon the diagram.

It may be noted at the outset that the diagram shows that the encased column is in every case considerably stronger ; the ratio of strength of plain column to that of encased column is 1·3 for a height or length of 2 ft., and is rather more for a length of 20 ft.

The vertical intercept between the two curves represents the increase in strength due to the presence of the concrete, and it will be seen that this is practically constant.

Compression tests were made of the concrete used for casing each column ; these tests were made for 6-in. cubes and for 8-in. diameter cylinders 16 in. long.

The average results after a period of 60 days were :—

	lb. per sq. in.
Ultimate compression strength for cubes	1,850
Ultimate compression strength for cylinders	1,220

The total area of the cross-section of the encased columns was 120 sq. in., and the area of the steel 13 sq. in. A very interesting result of the tests is that the increased strength of the encased columns agrees almostly exactly with that given by the compression tests of the cylinders, and is almost exactly two-thirds of that given by the cube tests.

As a result of the tests the authors state among their conclusions that : "A basis for design which seems rational is to determine the strength of the steel column by the use of the column formula for the $\frac{l}{r}$ of the steel columns, and then to consider the concrete of the core section (without reference to the length of the column for any length ratio, say a length of 15 diameters) to have a stress value proportional to the strength of the plain concrete, say, two-thirds of the cube strength. A suitable factor of safety would, of course, be somewhere applied."

The above tests were made at 60 days, and at the end of six months the strength of the encased columns would clearly have increased considerably further with the increased strength of the concrete.

BEAMS.

It does not appear to be easy to find test results upon the strength of beams encased in concrete, although many firms specialising upon filler-beam floors have probably conducted tests the results of which would be valuable at this juncture.

It is, however, fairly well-known that ordinary I-beams when tested for strength fail by buckling of the compression flange, and many specifications allow for this by specifying the extreme fibre stress for beams as a function of the ratio of the breadth of flange to the unsupported length of the flange. If the beam is encased in concrete, failure due to buckling or twisting cannot occur, and the beam must therefore be stronger than if the beam were not so encased.

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* University of Illinois Experimental Bulletin, No. 56, 1912.

This is very generally accepted, and many designers allow filler beams in concrete to be designed for a maximum fibre stress of 10 tons per sq. in. instead of the usual figure of 7·5 tons per sq. in.

The writer published some years ago a theoretical treatment of beams encased in concrete as a variation of the ordinary reinforced concrete beam formula making allowance for the moment of inertia of the beam section, and Mr. W. Cyril Cocking has given in his book, *Calculations for Steel Structures*, a set of tables for filler-beam floors based upon this theory.

There are also available accounts of the strengthening of railway-bridge girders by casting a concrete casing around them, and the experience with these should provide some useful information to guide us in this matter.

If any readers of *Concrete and Constructional Engineering* have available particulars of tests of steel beams encased in concrete they would be very useful in enabling engineers to arrive at a rational method of allowing in their design for the considerable increase in strength that must arise in practice from encasing the steel in concrete.

WAREHOUSE WITH GIRDERLESS ROOF AND FLOORS.

THE Harbour Authorities at Malmö have recently erected a large three-story warehouse, the floors and roof of which are without girders. The building is 400 ft. in length and 140 ft. in width at ground level. Its frontage is arranged step-wise, so that the cranes can deliver their loads direct on to any one of the three stories. The building is illustrated on p. 509.

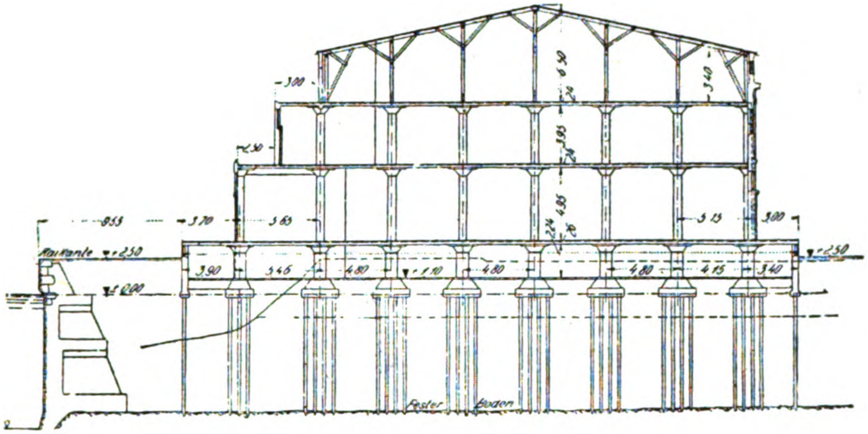
The building is erected on piles and is of customary construction with the exception of the floors, which are unusual in character so far as European buildings are concerned, though such " mushroom " roofs and floors are extensively used in the United States. The roof was designed by Dr. F. Emperger, of Vienna, and the calculations confirmed by Professor Forssell of Stockholm.

The pillars are cylindrical ; those on the ground floor are 9 in. in diameter, and those on the upper floors 8 in. in diameter. The reinforcement in the floors is composed wholly of round bars 0·56 in. diameter. The first floor was tested to 50 per cent. more than the calculated load, i.e., with 665 lbs. per sq. ft., but the largest deflection was only 0·08 in. Provision was made for horse-drawn wagons to be used in some parts of the building.

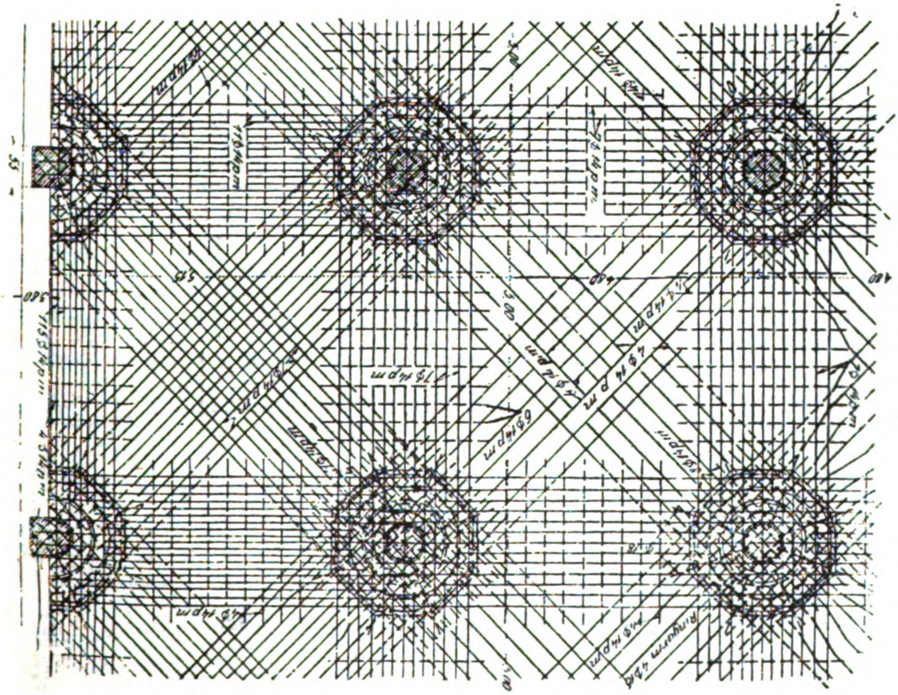
The simplicity of the shuttering is one of the great advantages of this form of roof, as it can be removed three days after the concrete has been placed and then used for other parts of the structure.

The walls extend throughout the whole height of the building without any interference by floor girders. The pillars and the under-parts of the floors are not plastered, but white-washed. The building required 500 tons of reinforcement and 8,000 cu. yds. of concrete.

The mushroom type of roof has the following advantages : (1) Only a simple and cheap shuttering is required, and this can be removed so quickly that it may be used repeatedly ; (2) the amount of work required for the shuttering is very small ; (3) all unnecessary height in the buildings is avoided ; (4) the total cost of construction is less than with other types.—*Beton u. Eisen.*



Section of Part of Warehouse.



Plan of One End and One Centre-Field in the Upper Floors.

WAREHOUSE WITH GIRDERLESS ROOF AND FLOORS. (*See p.* 508.)

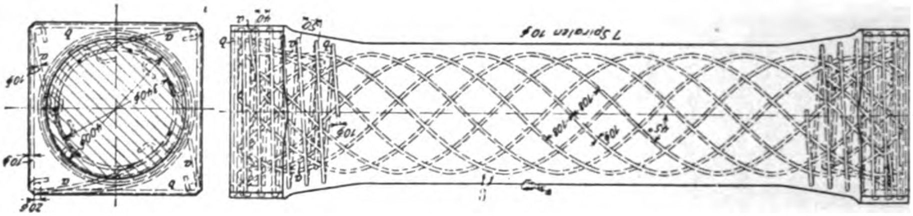


FIG. 5.

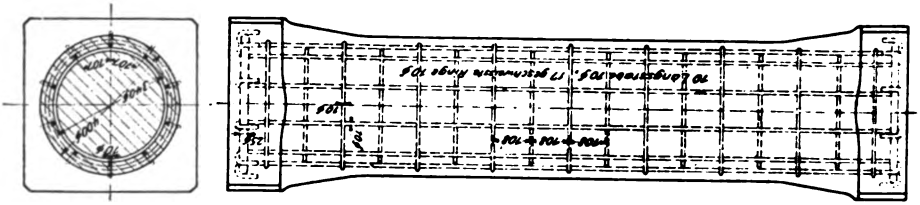


FIG. 4.

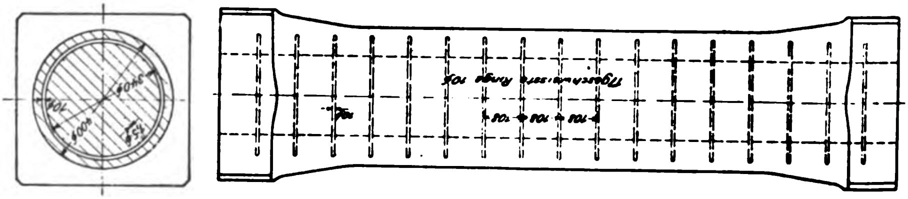


FIG. 3.

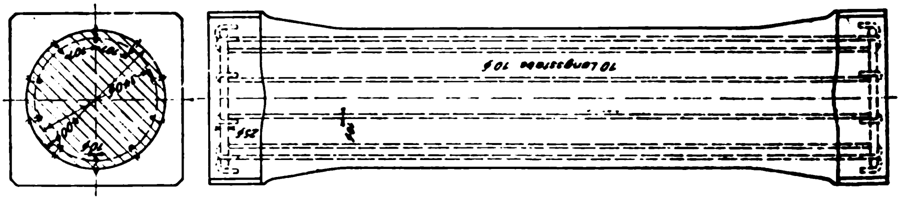


FIG. 2.

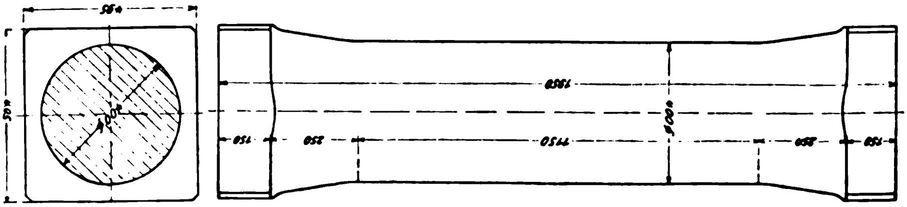


FIG. 1.

THE BEHAVIOUR OF CONCRETE UNDER THE ACTION OF TORSION. (See p. 511.)

THE BEHAVIOUR OF CONCRETE UNDER THE ACTION OF TORSION.

In this booklet * the authors have set themselves the problem of examining and explaining the behaviour of concrete, plain and reinforced, under the action of torsion, the first part of the book being devoted to the description of the experiments and tests carried out by Otto Graf, and the latter part containing a statical examination and explanation of the test results by Professor Mörsch. The tests were planned by the latter author, with a view to arriving at satisfactory conclusions regarding the resistance to shear of bodies of circular or annular section, and to obtain a clear insight in the shear problem generally, as it was not considered that previous experiments on this subject had furthered the knowledge of these problems sufficiently. In former experiments there has been some difficulty in making the test results agree with the conclusions arrived at through the usual calculations, dealing, as we are, with materials in which stresses and deformations are not exactly proportional, and where the modulus of elasticity is not the same for tension as for compression. To arrive at really satisfactory results in this respect an elaborate and most carefully-arranged series of tests was carried out at the Testing Laboratory of the Technical High School at Stuttgart, with a thoroughness and minuteness which have resulted in some very interesting and instructive observations being placed on record. The tests deal with both plain and reinforced concrete cylinders, the intention being to test the comparative value of various forms of reinforcement for the cylinders in relation to those without reinforcement. The reinforcement took the shape of longitudinal bars only, annular bars only, longitudinal and annular bars combined, and bars spirally curved at about 45 deg. to the cylinder axis. (See *Figs.* 1 to 5.) The cylinders were subjected to pure torsion, gradually applied, and careful readings were taken of the angular deformations together with any variations in the dimensions axially or diametrically. From the tests it appeared that the cylinders reinforced with only longitudinal or only annular bars behaved almost like the plain concrete cylinders, so that the value of this form of reinforcement may be said to be *nil* for all practical purposes. This bears out the contention of Professor Mörsch, made in 1907, that reinforcement at right angles to the shear force cannot possibly help to resist the shear with its own sectional shear area; and although a slight help is received against axial deformation of the cylinder, the longitudinal bars are unable to stop the bulging of the cylinder body after the first cracks have appeared. Likewise, the annular reinforcement, although able to restrain the bulging, is unable to stop the longitudinal deformation, and only when supplemented by longitudinal reinforcement do the annular bars become effective, so that by combined reinforcement a substantial increase (about 50 per cent.) in ultimate resistance is obtained, although the first appearance of cracks is not delayed to any considerable extent. Only by the use of spiral reinforcement at about 45 deg. to the cylinder axis are both the crack formation and the ultimate breaking of the test body arrested in a satisfactory manner, the ultimate strength being about three times that of a cylinder without reinforcement.

Test Body . . .	*Fig.* 1	*Fig.* 2	*Fig.* 3	*Fig.* 4	*Fig.* 5
Weight of Reinforcement in lb.. . .	0	49	28·5	77·5	51·5
Max. Torsion Moment in ft. lb. . . .	17,100	17,500	18,350	27,700	51,000

The tabulated results of all the tests are clearly and carefully arranged, giving the deformations, both elastic and permanent, at every step in the application of the torsion load, while a set of photographs and diagrams give full and chronological details of the formation of cracks during the progress of the test, the cracks being numbered in the order in which they appear.

While in the case of the plain cylinders and those reinforced as *Figs.* 2 and 3, the destruction of the test body practically occurs on the appearance of the first crack, the reinforcements in *Figs.* 4 and 5 have the effect of delaying the destruction for some considerable period after the first crack has appeared, as will be seen by referring to *Figs.* 6 to 9, which show the progressive formation of cracks at four stages of the load application for test bodies reinforced as *Fig.* 4, and in a still more marked degree in the case of bodies reinforced with spirals, as shown in *Figs.* 10 to 13, which indicate the appearance of the cylinder during four stages of loading.

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* Verdrehungsversuche zur Klärung der Schubfestigkeit von Eisenbeton. By Otto Graf and E. Mörsch. (Berlin, Julius Springer, 52 pp., 105 illustrations; price 1*s.* 6*d.*)

In the latter part of the book Professor Mörsch has summed up the test results in an admirable manner, and in examining them from a statical point of view finds a satisfactory harmony between calculated and experimental values. Professor Mörsch deduces various interesting facts which throw a clear light on the behaviour of concrete under torsional stress, more particularly as regards the effect of the difference between the moduli of elasticity for compression and for tension. The results are of great value, not only for the concrete specialist, but for any engineer dealing with materials that have the same peculiarities as concrete in respect of the elasticity modulus, such as, for instance, cast iron, which in many ways behaves almost exactly like concrete.

For those readers who understand the language the book contains a very

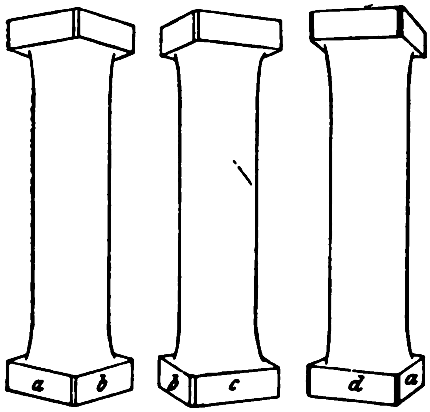


Fig. 6. M = 18,200 Ft. Lb.

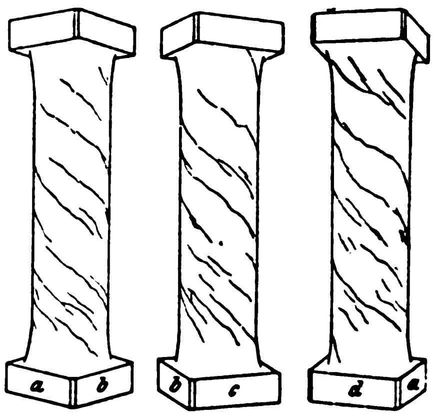


Fig. 8. M = 22,000 Ft. Lb.

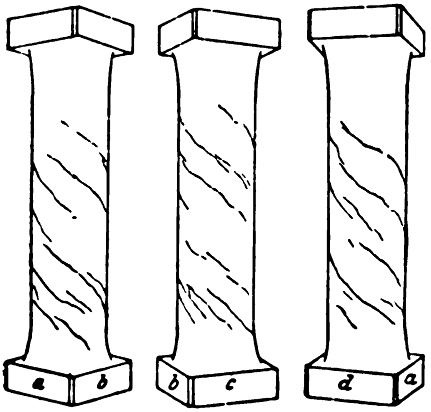


Fig. 7. M = 20,100 Ft. Lb.

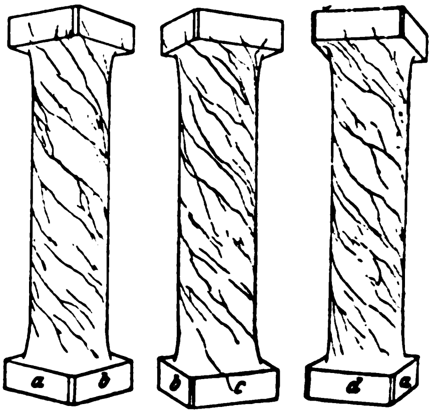


Fig. 9. M = 25,500 Ft. Lb.

interesting presentation of valuable facts and formulæ deduced from the most careful and painstaking investigation yet undertaken on this subject; the question of torsion stresses is one which the average designer most frequently leaves alone, and yet it is one that is met with in nearly every building structure. The curved balcony girder in a theatre or cinema is a striking case in point, but one need not go any further than to the ordinary lintel or wall-beam in a building, where the floor loads are applied on one side only. Whether the one-sided load is transmitted to the wall-beam through a simple slab or through a system of secondary beams, the eccentricity of the load must necessarily produce torsion in the supporting wall-beam. Even to the beams inside the building the same thing applies, in a less degree, as soon as the load is one-sided, but the stiffness and the monolithic nature of the structure is, as a rule, sufficient to take care of the torsional stresses without any special torsional reinforcement being needed.

R. N. S.

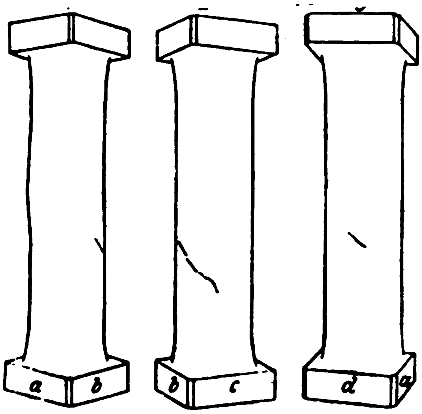


FIG. 10. M = 20,000 FT. LB.

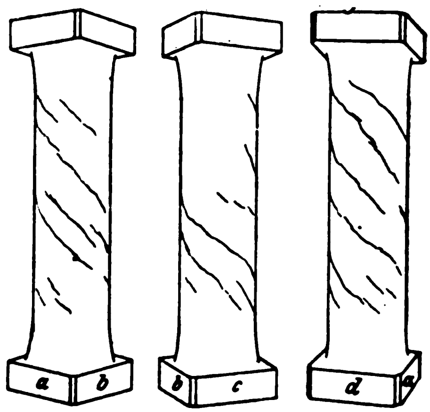


FIG. 11. M = 23,500 FT. LB.

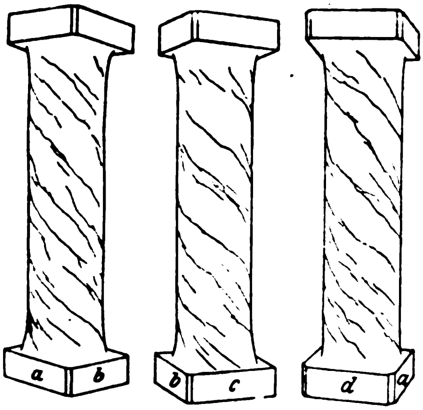


FIG. 12. M = 43,750 FT. LB.

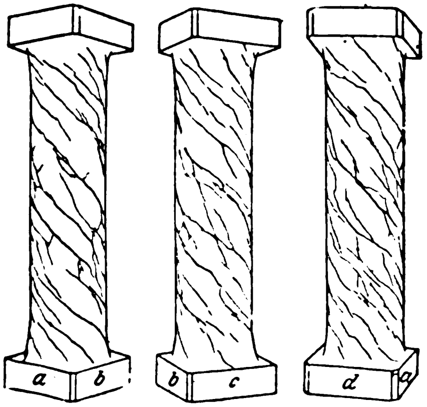


FIG. 13. M = 51,000 FT. LB.

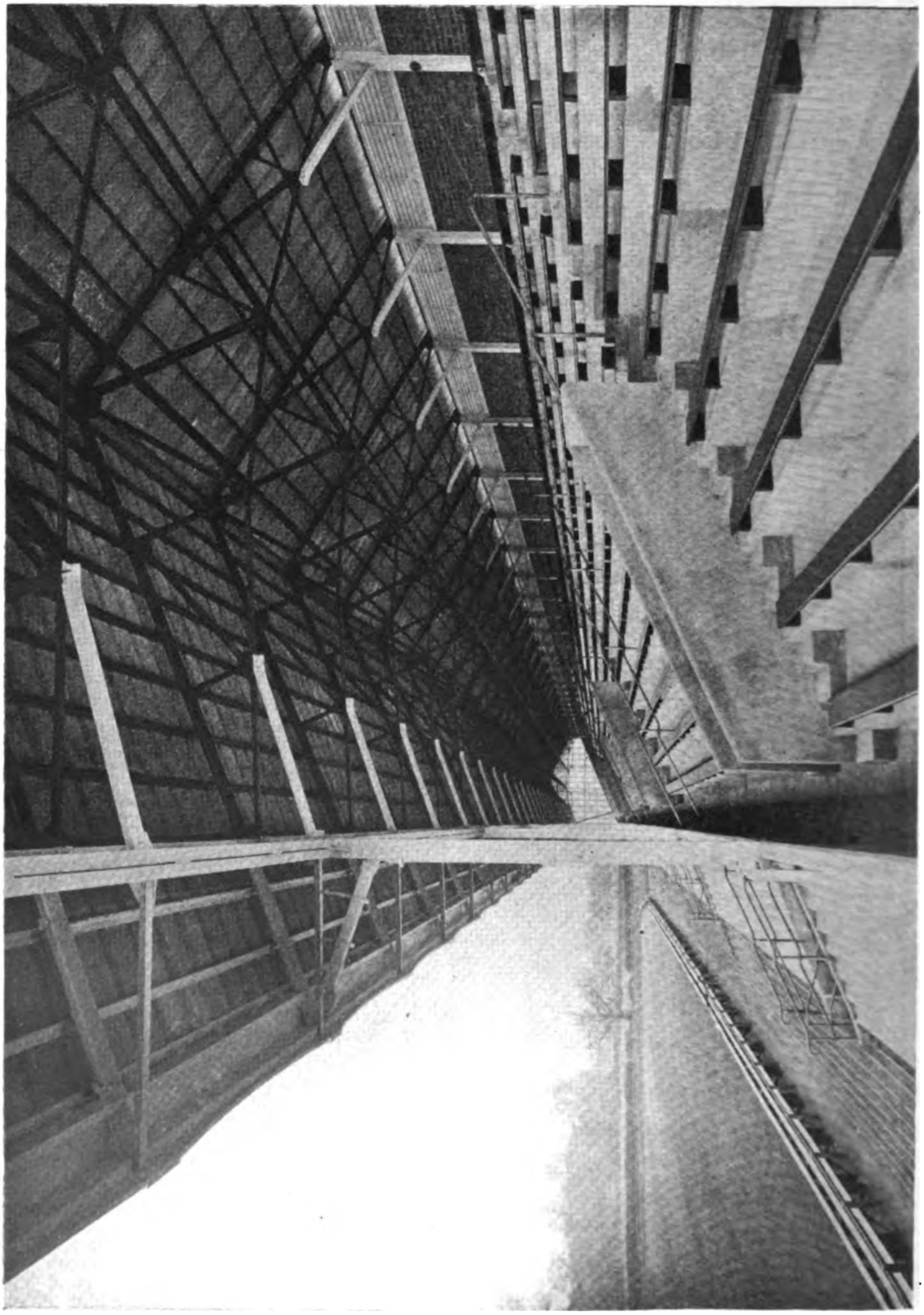


FIG. 2.—REINFORCED CONCRETE GRAND STAND AT BLACKHEATH. (See p. 515.)

REINFORCED CONCRETE GRAND STAND AT BLACKHEATH.

THE Blackheath Football, Cricket and Lawn Tennis Co., Ltd., are to be congratulated on obtaining such an excellent structure in the new reinforced concrete grand stand which has just been completed to accommodate spectators at their ground at Rectory Fields, Blackheath, S.E., which we illustrate in this issue. The stand was designed by Messrs. Walter Bridges & Co., consulting engineers and architects, of Parliament Street, S.W.1, and constructed under their supervision by Messrs. A. Roberts & Co., Ltd., builders and contractors, of 74, Earls Court Road, W.

The stand is 300 ft. long and 30 ft. wide, while the terraces in front, which extend along the whole of the length, form an additional forward extension of 25 ft. A roadway at the rear, 20 ft. in width, gives access to the stand, lavatories, offices, etc., underneath the stand, and also provides a parking space for cars.

The structure will accommodate 1,800 spectators seated in the stand and about 3,500 on the terraces in front. There are commodious and well-lighted offices and lavatories underneath the stand, and drainage has been laid for additional dressing-rooms, baths, etc., which it is proposed to erect at a future date.

The original stand, a timber structure of about half the size of the present building, was practically demolished by fire last summer, and the owners were well advised in replacing it with a permanent structure in reinforced concrete, which has a minimum of fire risk and requires the least possible maintenance.

The illustrations show the substantial proportions of the structure, the rigidity of the construction, and the dignified simplicity of treatment of the exterior. The æsthetic effect of the completed structure is happily not spoilt by an array of advertisements which it is customary to plaster over football stands nowadays.

A unique feature in the construction of the reinforced concrete sub-structure was in the pre-casting of the columns, which after being allowed to mature for three weeks were up-ended, rested on temporary timber supports, stayed and braced plumb (see *Fig.* 5, p. 520). The "blob" footings, bracings and ground ties were then cast *in situ*, after the attachment of the reinforcement to the bond bars already cast in the columns, as shown in *Fig.* 5.

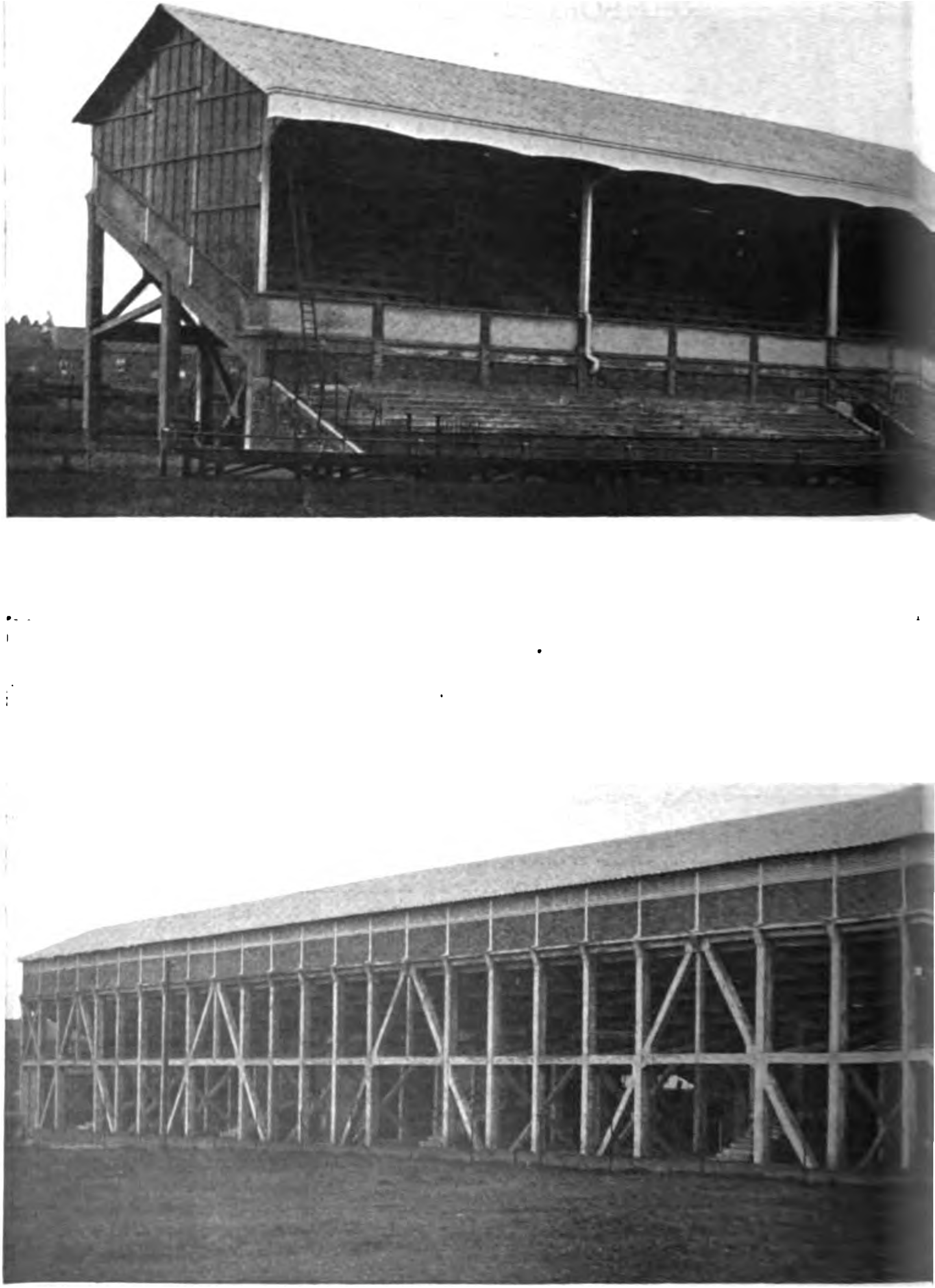
The columns are of various sections, from 10 in. by 10 in. to 14 in. by 14 in., dependent upon the unbraced length, the degree of fixity at the footings, and the load to be carried; they are arranged in three rows, approximately 15 ft. apart, with 10 ft. centre to centre between columns in each row.

All ground ties and cross-bracings are 8 in. by 8 in. in section, reinforced with four ¾-in. bars bound together with rectilinear binding $\frac{3}{16}$-in. in diameter at 3 in. centre to centre throughout.

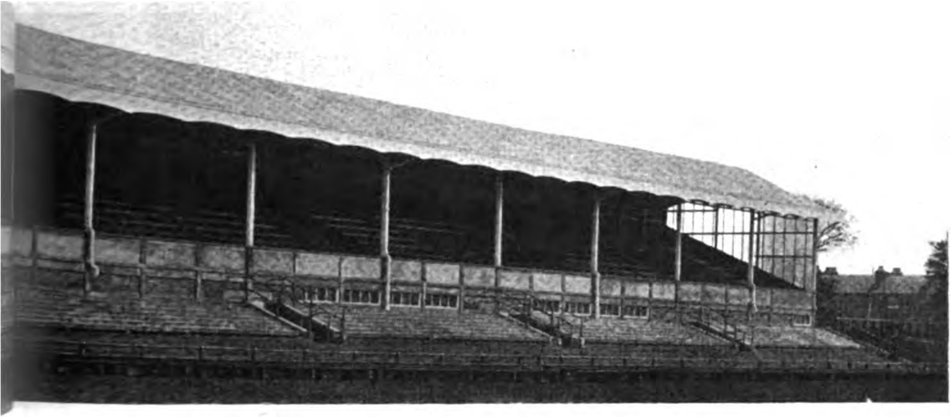
The treads of the steppings of the stand were made in 3 in. pre-cast reinforced concrete slabs, which were erected on to ledges formed in the pre-cast riser beams and provided with rebates 6 in. by ¾ in. at back and front, in which projecting expanded metal reinforcement from the risers was laid and afterwards grouted in. In this manner the advantages resulting from the use of pre-cast members were obtained without sacrificing to any great extent the continuity of monolithic construction.

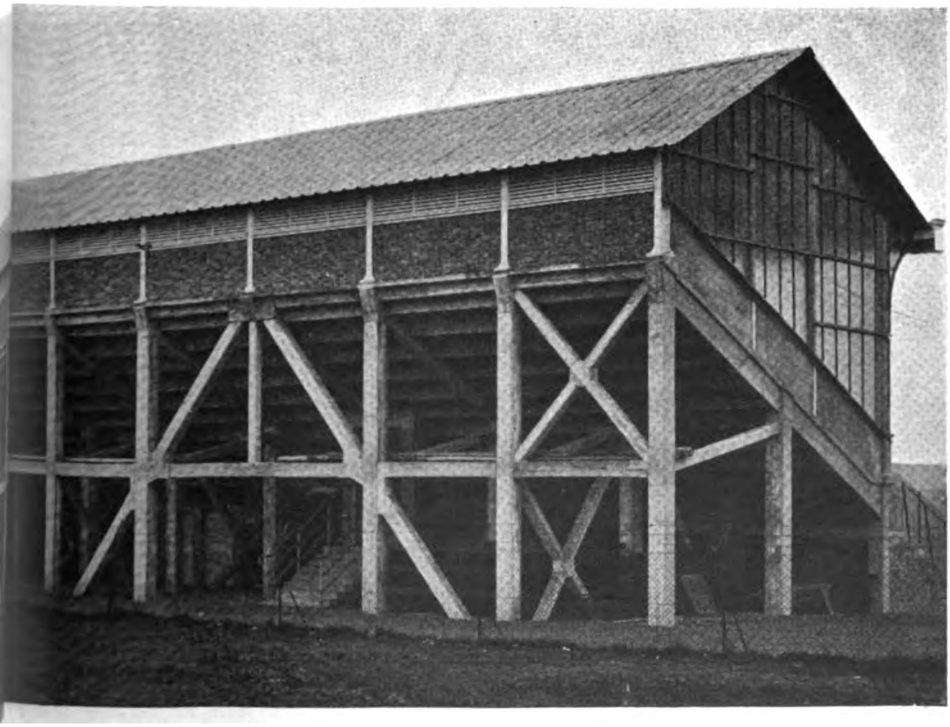
The formwork for the raker beams, which were cast *in situ* as the bonding member between columns and riser beams, had to carry the dead load of risers and treads in addition to the weight of the wet concrete of the beam itself. For this reason, and also to utilise the load-bearing capacity of the reinforced concrete columns, the underside of this formwork was made in the form of substantial inverted trusses in timber spanning the columns (see *Fig.* 6, p. 521). A similar truss was erected as a carrier under the centre of the span of the risers to take some of the weight off the end boxes.

Owing to the method of construction contemplated the raker beams were designed as of rectangular section, with equal steel top and bottom. Further, to avoid congestion of steel where the reinforcement of the risers overlapped within these beams, no bent up bars were used, the main reinforcement passing in one length from back to front of the stand.



REINFORCED CONCRETE GRAND STAND AT BLACKI





EATH. FIGS. 1 AND 3.—FRONT AND REAR VIEWS.

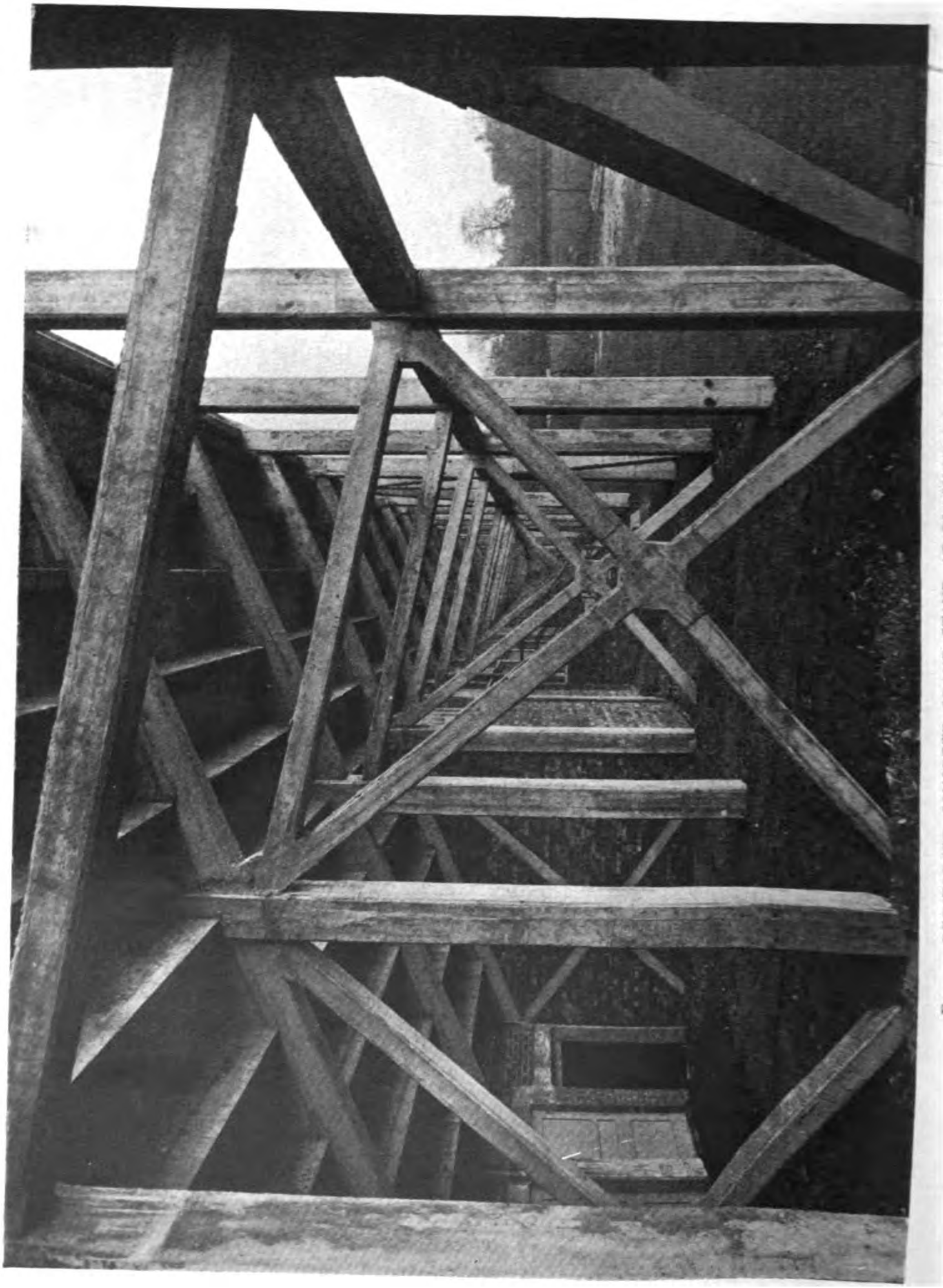


FIG. 4.—REINFORCED CONCRETE GRAND STAND AT BLACKHEATH. (See p. 515.)

Ample shear resistance was provided by the tension in the vertical stirrups, combined with the diagonal compression in the concrete.

The riser beams were reinforced with straight bars top and bottom, the expanded metal web reinforcement being bent round the bars and made to project beyond the sides of the risers to bond with the treads, as already described.

Mild steel bars for a calculated maximum stress of 16,000 lb. per sq. in. were used throughout, and the panels of the front parapet and curtain walls around stairwells were formed with Portland cement plaster on " Self-sentering " expanded metal.

It will be seen from *Fig.* 3 that the bracing is arranged in the form of a pyramid round each of the five staircases. The front diagonal braces are 8 in. by 8 in. in section, those at the back being 11 in. by 11 in.

The pyramid base is 40 ft. by 30 ft., and the ten columns on the perimeter are joined by 8 in. by 8 in. ground ties.

The arrangement of the bracing in the fore and aft direction can be seen from *Fig.* 4, which shows the concrete as it came from the moulds and before the surface was finished.

The steelwork of the superstructure was manufactured and erected by Messrs. James Allan, Senr., & Sons, of Glasgow, and is shown on *Fig.* 2. Trusses are spaced 10 ft. centres, and each is supported at the back of the stand on short steel stanchions of 12 in. by 5 in. British Standard section well anchored, so as to resist the whole of the horizontal wind stresses.

The trusses at the front are supported on light lattice girders of 30 ft. span, which are bolted to 7 in. by 7 in. stanchions arranged so that the rectangular down pipes fit into the bosom of the joists.

Fig. 2 also shows the arrangement of the seats, and the backs provided to the seats in the middle portion of the stand.

The risers of the reinforced concrete steppings to the stand are 1 ft. 3 in., and the width of tread 2 ft. 9 in., this latter being somewhat unusually large, but providing much greater comfort for the spectators. The actual proportion of tread to riser was ascertained after a careful study of the levels of the field of play, and the slope calculated so that all spectators have an uninterrupted view of the near touch-line.

The front valance boarding was kept sufficiently high to enable the back row of spectators to see the ball throughout its trajectory when punted near the stand.

The terraces in front of the stand, which are also shown in *Fig.* 2, are constructed of steps 6 in. by 15 in., the lower nine being formed by a solid bank riveted with timber and the upper seven steps of reinforced concrete spanning 9 in. brick walls at 10 ft. centres.

An independent test made on specimens of concrete taken from batches mixed for the actual job showed excellent results. Three 6 in. cubes were tested, and gave the following crushing strengths per sq. in. at 29 days: 3,940 lb., 4,020 lb. and 4,010 lb., an average of 3,990 lb. The proportion of the concrete was 1 part Portland cement, 2 parts sand, and 4 parts ballast, graded so that the largest particles passed a ¾ in. ring.

We understand that cost data are not available for publication, but we are interested to learn that a well-known firm of contractors obtained permission to offer a tender for an alternative construction in steel and that their tender was not by any means the lowest.

Concrete Houses at Belfast.

THE Belfast Corporation has decided to carry out an extensive housing scheme in coloured concrete on the same lines as the Liverpool Corporation housing scheme illustrated and described in our issue for January last. The decision to adopt concrete construction followed a visit to Liverpool of a deputation from the Belfast Corporation, who made a thorough inspection of the concrete houses which have been erected there under the direction of Mr. F. E. G. Badger, the Director of Housing to the Liverpool Corporation. As on the Liverpool scheme, " Winget " machines are to be used for the manufacture of the concrete blocks, and we understand that the Liverpool Corporation has offered to place at the disposal of the Belfast Corporation a number of men experienced in this class of work to train local workers on the Belfast scheme.

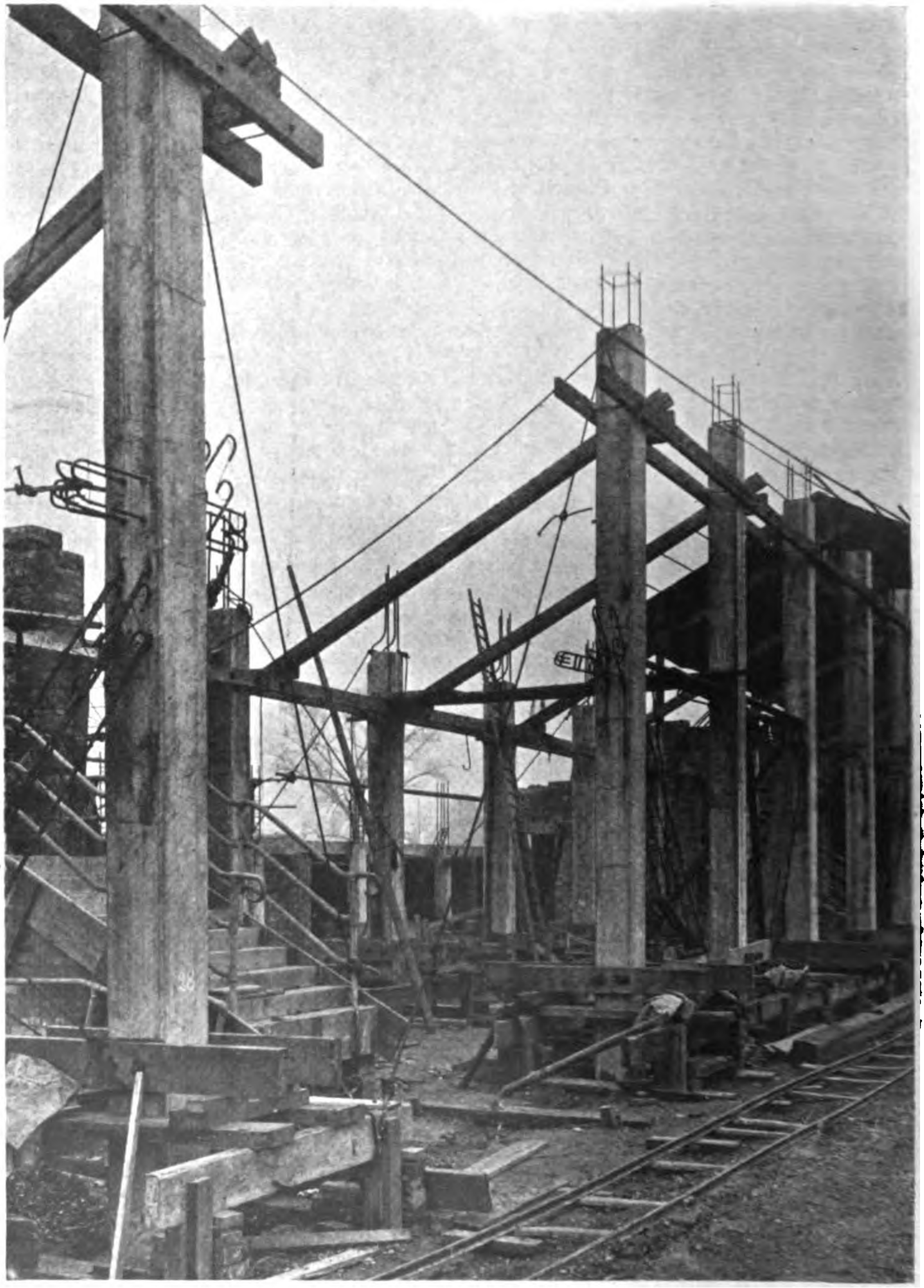


FIG. 5.—REINFORCED CONCRETE GRAND STAND AT BLACKHEATH. (*See p.* 515.)

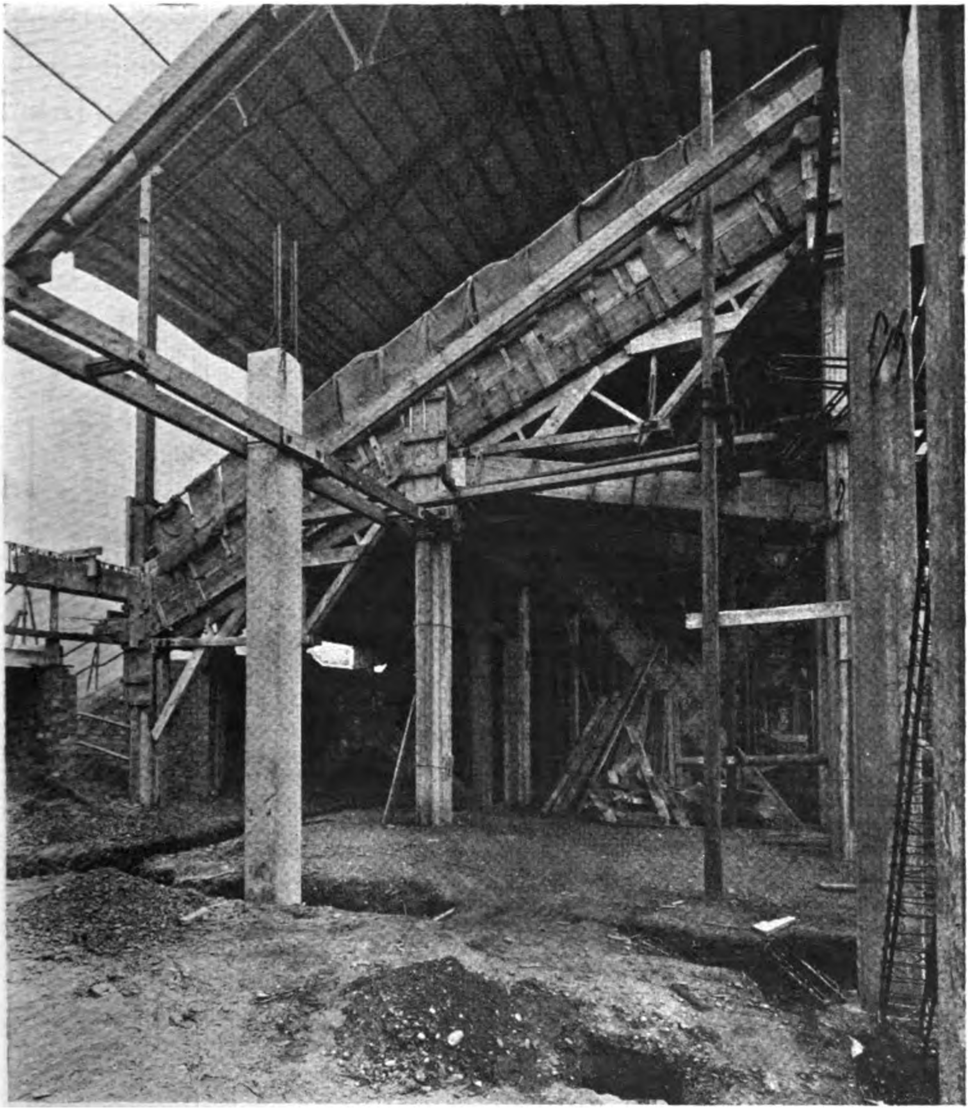


Fig. 6.—REINFORCED CONCRETE GRAND STAND AT BLACKHEATH. (See p. 515)

THE EFFECT OF SULPHUR-CONTENT ON PORTLAND CEMENT.

A PAPER communicated to the Institution of Civil Engineers by Professor F. C. Lea, O.B.E., D.Sc., M.Inst.C.E., and Mr. R. E. Stradling, M.C., Ph.D., Assoc. M. Inst. C.E., entitled "Some Experiments on the Effect of High Sulphur-Content on Portland Cement," states that specifications for cement limit the allowable proportion of sulphur (as SO_3), and the question not infrequently arises as to whether cement should be condemned on account of the quantity of SO_3 present. The British Standard specification states that it shall not exceed 2·75 per cent. Manufacturers of cement add gypsum during the process of manufacture, the amount so added depending to a certain extent upon the season of the year and the setting-time required.

The object of the experiments was to determine the effect on the strength and soundness of cement of varying the sulphur-content by the addition of gypsum; also its effect on steel embedded in the cement. The experiments have been carried out in the University of Birmingham, with cements specially made for the purpose. No attempt was made to produce a commercial cement that would satisfy the British Standard Specification in all details.

Starting with a clinker containing about 2·5 per cent. sulphur (as SO_3), other cements were made by the addition of proportions of gypsum, ranging from 1 to 10 per cent. Thus the percentage of sulphur was approximately as follows :—

Cement Sample No.	Gypsum added.	SO_3.	Cement Sample No.	Gypsum added.	SO_3.
	Per Cent.	Per Cent.		Per Cent.	Per Cent.
1	0	2·5	7	6	5·0
2	1	2·9	8	7	5·4
3	2	3·4	9	8	5·8
4	3	3·8	10	9	6·1
5	4	4·2	11	10	6·5
6	5	4·6			

With these special cements five types of specimens were made, and the following tests and observations were carried out :—

(*a*) Le Chatelier tests for soundness;

(*b*) Tensile strength: neat cement;

(*c*) Tensile strength: cement and sand (1 to 3);

(*d*) Compression strength tests on 4-in. concrete cubes;

(*e*) The effect of sulphur on steel embedded in concrete cylinders.

The 4-in. cubes and the cylinders were suitable for observing the behaviour of the cements as far as expansion and cracking with time were concerned. The details of these experiments and the results so far obtained are as follows :—

TABLE I.

Cement Sample No. (All mixed with 26 per Cent. Water).	Le Chatelier Expansion.		
	(1)	(2)	Mean
	Mm.	Mm.	Mm.
1	1·0	1·0	1 0
2	0·5	0·5	0·5
3	1·0	1·0	1·0
4	1·0	1·0	1·0
5	0·8	0·7	0·75
6	1·0	1·0	1·0
7	0·5	0 8	0·65
8	1·4	0·5	0·95
9	1·0	0·9	0·95
10	1·0	0·6	0·8

(*a*) LE CHATELIER TESTS FOR SOUNDNESS.—Two specimens of each sample included in Table 1 were made, and the tests were carried out as specified by the British Standard Specification for Portland cement. As will be seen from this

Table, no appreciable expansion was obtained in any case; yet, as will be shown later, some of the concretes were undoubtedly unsound, expanding and cracking badly with time.

(*b*) and (*c*) TENSILE STRENGTH: NEAT CEMENT AND CEMENT AND SAND.—These briquettes were of the type specified by the British Standard Specification, and required about 26 per cent. of water to obtain a suitable consistency for the neat cement, and 9 per cent. of water for cement and sand mixture (1 part cement, 3 parts Leighton Buzzard sand). They were made and stored as required by the British Standard Specification, and the breaking strengths (average of three briquettes at each age) were as given in Table II.

The strength begins to diminish only when the addition of gypsum has reached a total of 5–6 per cent., corresponding with 4½–5 per cent. of SO_3. There appears to be a slight increase in strength up to about 4 per cent. added gypsum (corresponding with just over 4 per cent. SO_3).

TABLE II.—TENSILE STRENGTH: NEAT CEMENT AND CEMENT AND SAND.

Cement Sample No.	Average Strength.			
	7 Days.		28 Days.	
	Neat.	Cement and Sand.	Neat.	Cement and Sand.
	Lbs.	Lbs.	Lbs.	Lbs.
1	442	189	421	217
2	497	194	478	260
3	513	197	480	282
4	466	206	499	268
5	529	208	516	290
6	486	175	482	260
7	314	124	442	246
8	235	56	421	185
9	220	47	344	77

(*d*) CONCRETE CUBES FOR COMPRESSION TESTS.—Owing to shortage of storage-accommodation it was not possible to make blocks from all the samples of cement, and blocks were therefore made only of cements numbered 1 to 5, 7, 9, and 11. Eight cubes of concrete were cast with each of these cements, giving a total of 64 cubes. The concrete consisted of 1 part cement, 2 parts sand (from Cannock Chase), 4 parts quartzite (from Nuneaton), and was cast into 4-in. cubes and 4½-in. dia. cylinders. Half of these cubes were stored in air in a dry room, and the other half in water. At the end of 11 months the blocks of the first set were broken. The results are given in Table III.

TABLE III.

Cement Sample No.	Compressive Strength.		Remarks.
	Stored in Air.	Stored in Water.	
	Ton per sq. in.	Ton per sq. in.	
1	1·14	1·89	
2	1·04	1·73	
3	1·23	1·60	
4	1·06	1·60	
5	1·15	1·54	
7	0·99	1·80	
9	0·74	1·44	Shows distinct cracks.
11	0·71	0·297	Badly split and cracked.

Unfortunately only one specimen of each was available for test, as the water-storage accommodation is not sufficient for a larger number. It is proposed to break the remaining cubes at intervals of 2, 5, and 10 years. The number of cubes still unbroken is 48, of which 24 are stored in air and 24 in water.

The specimens from samples 9 and 11 were cracked at the end of 1 month. At the age of 11 months no appreciable increase in the disintegration was apparent over that found at 1 month. The blocks made from sample 11 were very badly cracked and broken, whilst those made from sample 9 had only cracks just appearing. These again were visible at 1 month, and did not appear to have increased on further ageing. Decrease in compressive strength takes place when 6–7 per cent. of gypsum is added (5–5½ per cent. SO_3).

(*e*) CONCRETE CYLINDERS CONTAINING STEEL.—Eight cylinders of each of the mixtures used for the cubes were also cast and were stored under the same conditions as the compression cubes. Forty-eight of these are still unbroken, 24 of which are stored in water and 24 in air. Two pieces of steel were embedded in each cylinder. One was brightly polished and the other one was rusted.

After an interval of 11 months a cylinder of each sample was broken open, and the steel bars were examined. In no case was it possible to say that definite rusting had taken place due to the action of the concrete, or due to water and air getting into the concrete. Slight brownish-yellow skins and occasionally rust-markings were found on some of the bright bars.

SUMMARY AND CONCLUSIONS.

(i) High percentages of gypsum reduce the strength of cement, cement mortar, and concrete.

(ii) At the end of 1 month concretes with cement containing over 6 per cent. SO_3 were cracked, and with 6½ per cent. SO_3 were badly cracked.

(iii) No obvious disintegration during a period of 11 months was observable on any samples of cement or concrete with a cement having a lower percentage of SO_3 than 6 per cent.

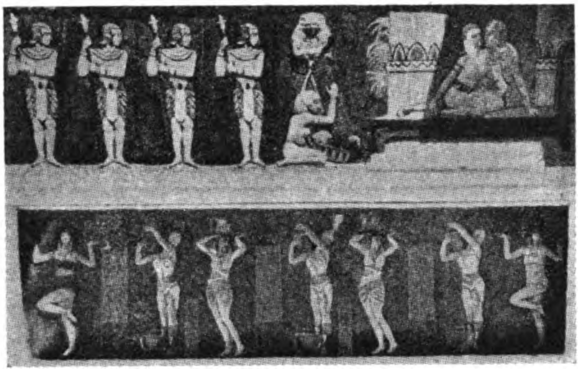
(iv) The Le Chatelier test for soundness fails to detect probable unsoundness due to high gypsum-content.

(v) In none of the samples used was it possible to detect any effect on steel, either bright or rusted, which could be directly attributed to the gypsum content.

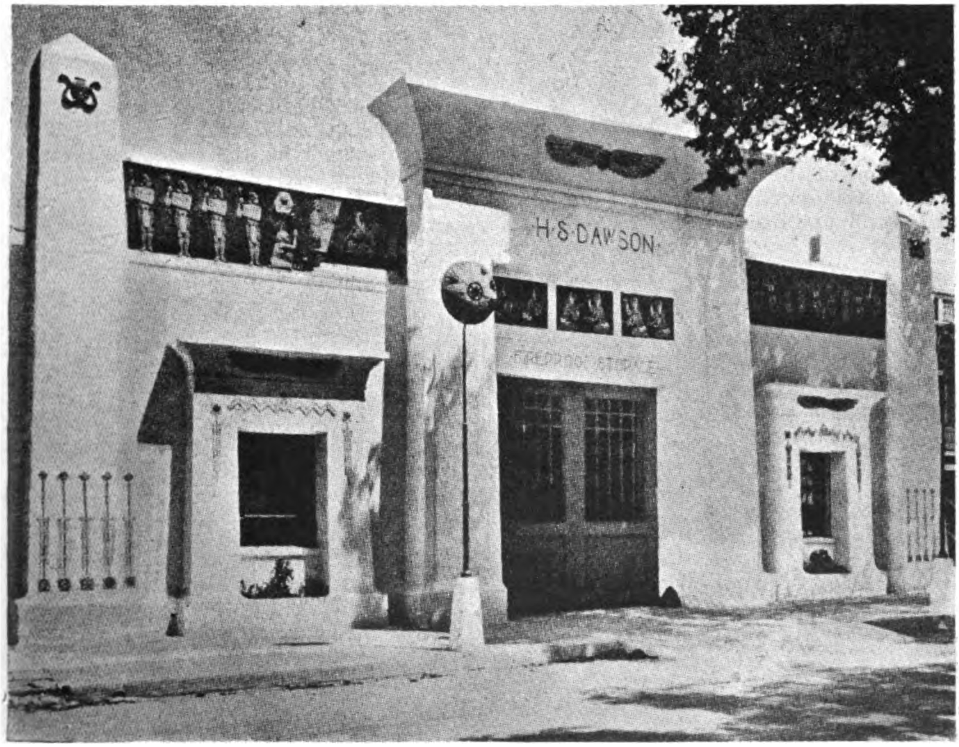
The conclusion as to soundness is clearly of the greatest importance, and when analyses of the cements are not directly made, or supplied by the makers, it is suggested that blocks should be kept under observation for at least 1 month. If these do not show signs of cracking after that period, there will probably be little fear of cracks developing later.

CONCRETE WAREHOUSE IN CALIFORNIA.

THE illustrations on p. 525, for which we are indebted to our American contemporary *Concrete*, show the extent to which the decorative possibilities of concrete are being taken advantage of in the States. The frieze and other details are all of pre-cast concrete coloured with enamels, ground and mixed in one-half turpentine and one-half spar varnish for a binder; for a second coat, the same material, using 1 part of turpentine to 1 gal. of varnish. On this 2-coat base the final colour work was done, the colours being mixed in spar varnish. The architect states that the colours have been highly satisfactory as to durability.



DETAIL OF FRIEZE.



[Mr. Glenn Allen, Architect.

CONCRETE WAREHOUSE IN CALIFORNIA. (*See p.* 524.)

FERRO-CONCRETE GAS-HOLDER TANK AT DUNDEE.

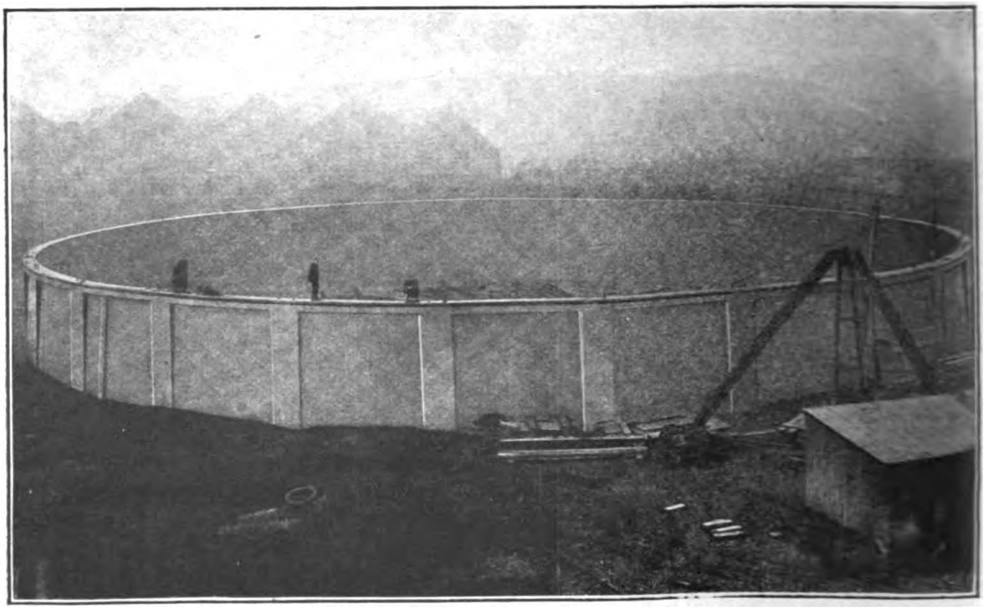
The illustrations on this and the following pages show a reinforced-concrete gas-holder tank which has recently been constructed at Dundee Gas Works, under the direction of Mr. Alex. Yuill, engineer to the Gas Commissioners. It is believed to be the largest ferro-concrete gas-holder tank yet completed in this country, and has a diameter, measured inside the wall surface, of 172 ft. 6 in., while the depth of the wall is 35 ft. 6 in. Further dimensions are shown on the illustrations.

The walls and the central columns were carried down to a solid rock foundation, the floor of the tank being laid on a 6-in. filling of hardcore. In a circular wall of this magnitude the stresses due to the internal pressure of the water are taken up partly by the ring reinforcement and partly by the vertical units of the wall acting as beams fixed at the bottom and supported at the top, the tensile stresses in the ring reinforcement and the bending stresses in the vertical units being proportioned in such a way that the elongation, measured on the radius, due to the direct tension in the ring reinforcement equals at all points the horizontal deflection of the wall acting as a beam.

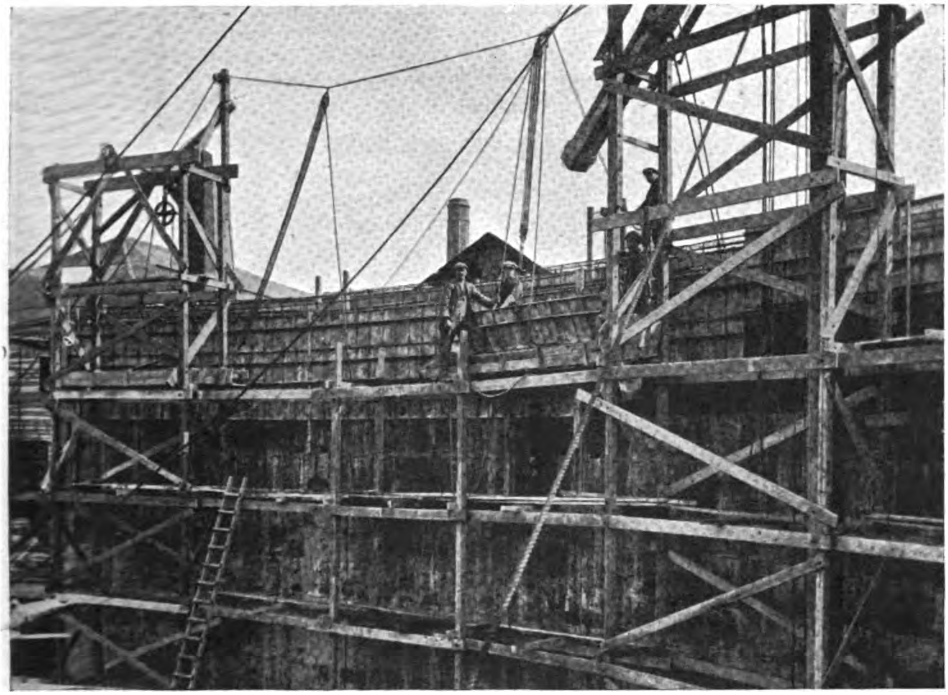
The reinforcement throughout consisted of plain round mild steel bars varying in diameter between ¼ in. and 1 in., and having an ultimate strength of 28–33 tons per sq. in. The concrete was a 1 : 2 : 4 mixture of Portland cement, clean washed river sand, and whinstone from local quarries crushed to ¾ in.–¼ in. gauge. The stresses in the steel and concrete were limited to 16,000 lb. (tensile) and 600 lb. (compressive) respectively.

The work was carried out with the aid of an "Insley" chuting plant and "Blaw" steel light wall forms, the mixing being done in two electrically-driven "Victoria" concrete mixers. The walls were coated internally with cement slurry brushed on, to ensure perfect watertightness, and the exposed surfaces were rubbed down and coated with a wash of cement to give them a smooth and even appearance. The photographs show the excellent finish obtainable with the use of the steel forms used. These were supplied by the Blaw Knox Co., of Pittsburgh, Penn., whose sole concessionaires in Great Britain are Messrs. Christmas & Walters, Ltd., of Caxton House, Westminster, S.W.1.

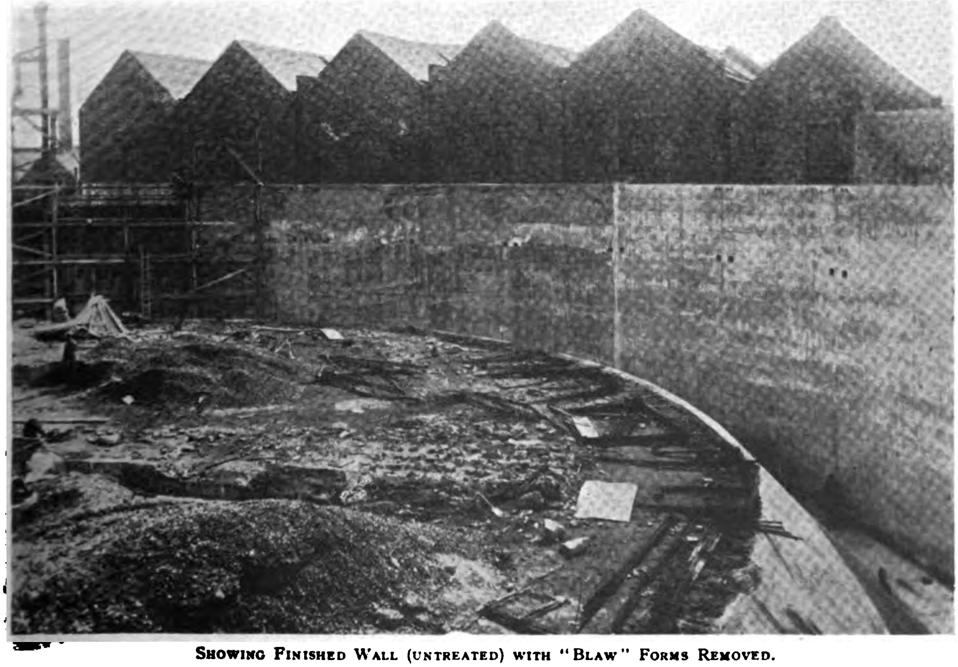
The contract for this work was placed with Messrs. K. Holst & Co., of 1, Victoria Street, London, S.W.1, who carried out the ferro-concrete work to their own design.



Reinforced Concrete Gas-Holder Tank at Dundee.

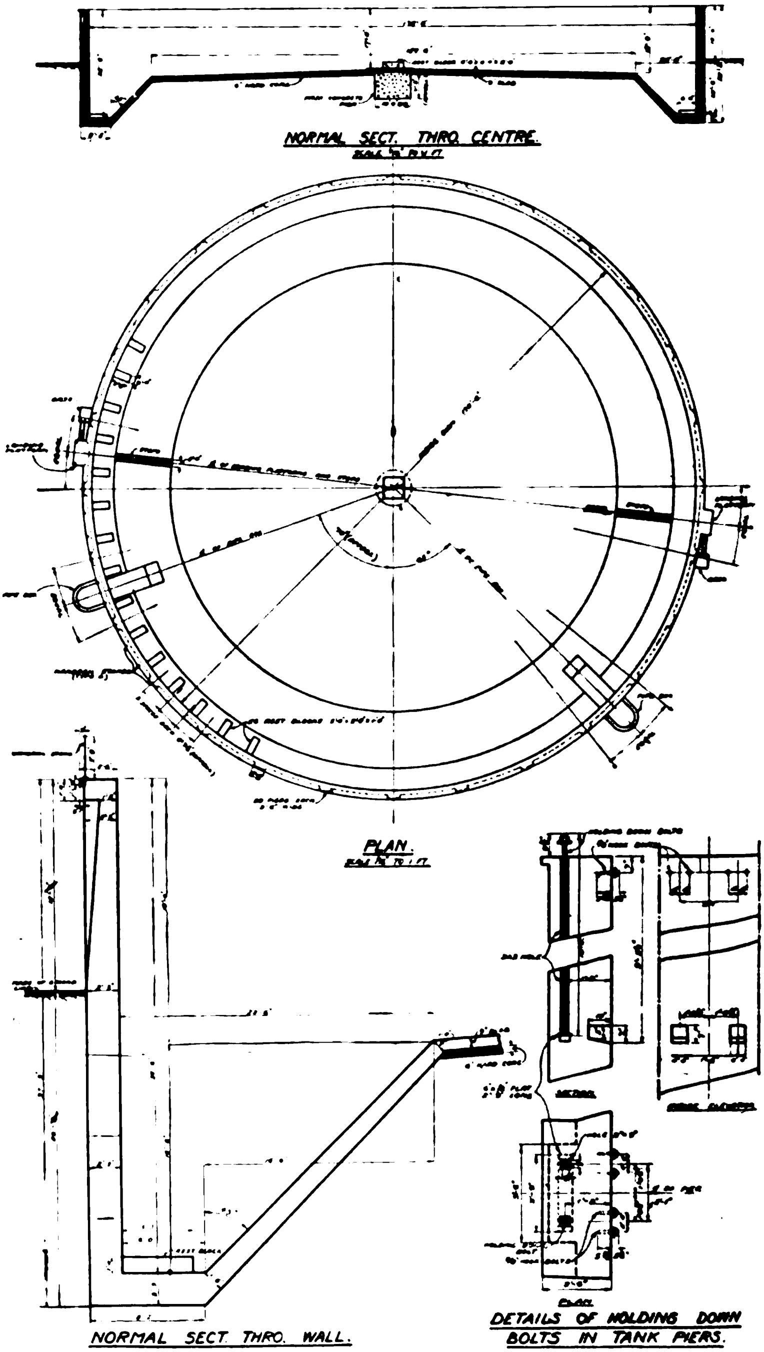


SHOWING STEEL "BLAW" FORMS.

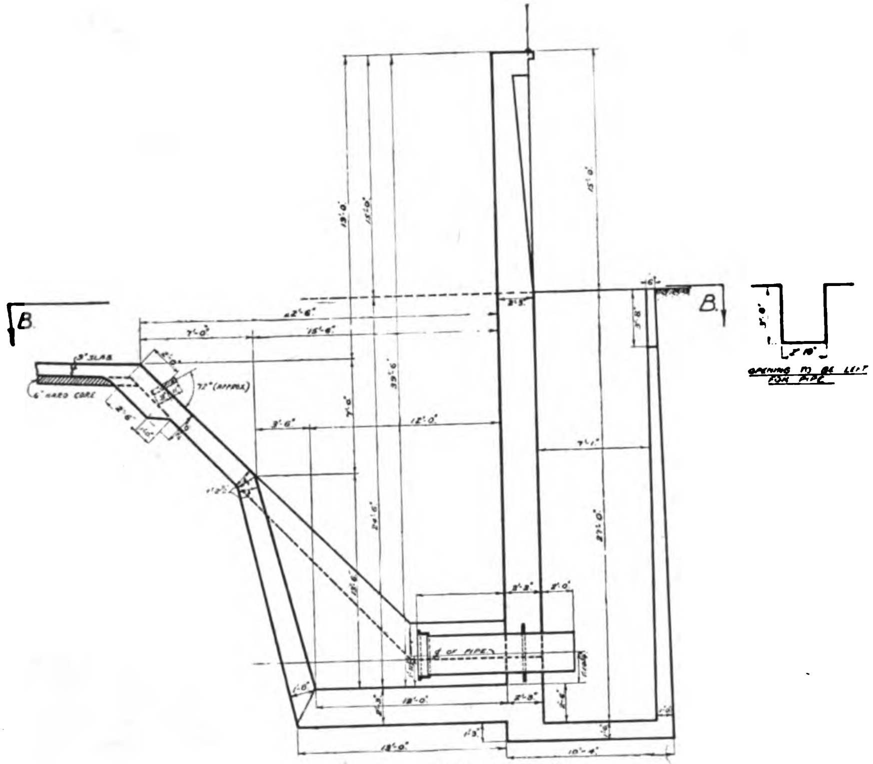


SHOWING FINISHED WALL (UNTREATED) WITH "BLAW" FORMS REMOVED.

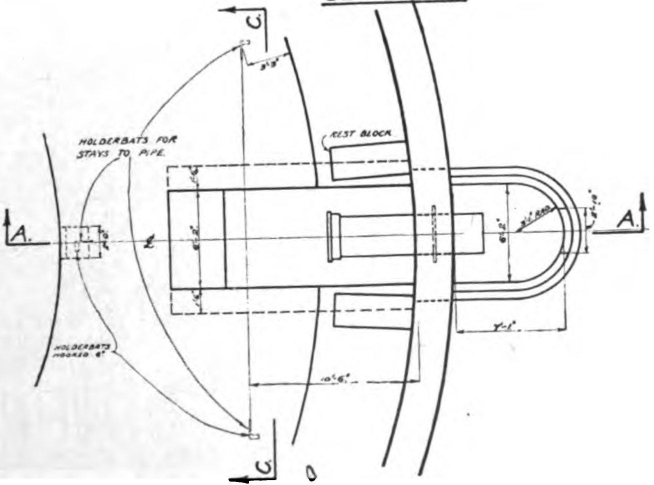
REINFORCED CONCRETE GAS-HOLDER TANK AT DUNDEE.


NORMAL SECT. THRO. CENTRE.
PLAN.
NORMAL SECT. THRO. WALL.
DETAILS OF HOLDING DOWN BOLTS IN TANK PIERS.

REINFORCED CONCRETE GAS-HOLDER TANK AT DUNDEE. (*See p.* 526.)



SECTION A-A.



SECTION B-B.

DETAILS OF PIPE BOX.

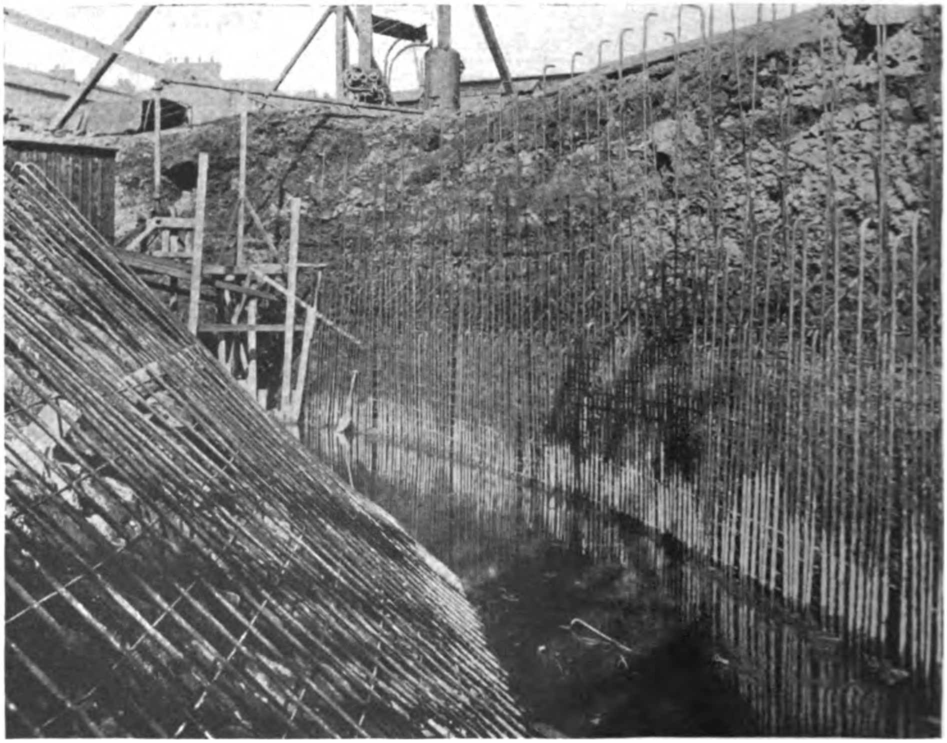
REINFORCED CONCRETE GAS-HOLDER TANK AT DUNDEE. (*See p.* 526.)

BOOK REVIEWS.

Cements, Limes and Plasters. By Edwin C. Eckel.
(Second Edition. Pp. xxxi + 655. London: Chapman & Hall, Ltd. Price 32s. 6d.)

THIS second edition of a work already well known to students of cement manufacture deals primarily with American practice, but its interest to readers of other nations is not appreciably restricted on this account. The largest section of the book is naturally devoted to Portland cement, and this follows the usual procedure of describing raw materials and their preparation, methods of burning and grinding, and concluding with the properties and testing of cement. There are, however, some features of this section worthy of special comment, the first being the exhaustive manner in which the raw materials for cement manufacture are dealt with. Each typical raw material is described from geological, chemical, and mechanical standpoints; methods of excavation are discussed, and numerous analyses are given, with copious references to other literature upon the subject.

Another feature to be commended is the presentation of costs of each stage of the manufacture, and the author has evidently had access to figures which are seldom available to the general public. The costs are based on prices current from 1910 to 1915, and by means of the table of average wholesale commodity prices from 1890 to 1920 it is possible to relate the basic costs of 1910–1915 to any other period; adopting this method, the author has performed a needed service by reducing the war and post-war prices of cement to a true and comparable price after allowing for the changed value of currency. The result shows that the 1918–1920 prices of cement in the United States were in reality lower than those of 1910–1915. On the subject of cement prices it is interesting to observe that the author takes the same line as British



REINFORCED CONCRETE GAS-HOLDER TANK AT DUNDEE, SHOWING REINFORCEMENT. (*See p. 526.*)

cement manufacturers in protesting that engineers, after demanding and paying extra for finely-ground cement, seldom make proper use of the improved quality, but employ the cement in the same mixtures as were considered proper when the cement was much coarser and correspondingly weaker.

Heat consumption and heat utilisation in rotary kilns form the subject of another chapter, and is one concerning which the dissemination of reliable information is always valuable.

In some respects the book has obviously suffered from inadequate revision of the first edition; for example, reports of 1905 and before are described as "recent," and the British Standard Specification for Portland Cement, which is set out in full, is the edition of 1905, which has been obsolete for thirteen years. The lack of sufficient revision becomes unfair to British cement manufacturers when it is implied that English cements leave a residue of 15 per cent. on the 100-mesh sieve, and when comparative tests of English, American, and German cements made in 1899, described as "quite representative," are included which show English cement to possess a tensile strength of less than 500 lb. per sq. in. at seven days and to be decidedly inferior to American and German cements. The average British cement of to-day leaves a residue of less than 5 per cent. on the 100-mesh sieve and has a tensile strength at seven days exceeding 700 lb. per sq. in.

One looks in vain for any but the briefest reference to certain recent developments in cement manufacture which have been particularly prominent in America, notably the application of waste-heat boilers to dry process rotary kilns, the method of mechanical packing of cement which is almost universal in the United States, and the substitution of flint pebbles by metallic grinding bodies in tube mills. In connection with the last mentioned, considerable space is given to the description of flint pebbles although these are fast becoming obsolete as a grinding body for cement manufacture.

Other sections of the book deal with the raw materials for manufacture and properties of plasters (viz., products of gypsum), limes, magnesia and oxychloride cements, natural cements, and slag cements. The chapters on lime are valuable, and one is led to wonder whether the inefficient slaking of lime as now conducted on English constructional jobs by ordinary labourers will be replaced by scientifically-prepared "hydrated lime" as is occurring in America.

As a compendium of information on modern binding materials used in constructional work the book is probably unique, and with its very extensive bibliography should be of particular value to the student, although one could wish that the revision had been more thorough and the somewhat numerous typographical errors had been corrected.

Structural Drafting and the Design of Details. By Carlton Thomas Bishop.

(London: Chapman and Hall. Price 25s. net.)

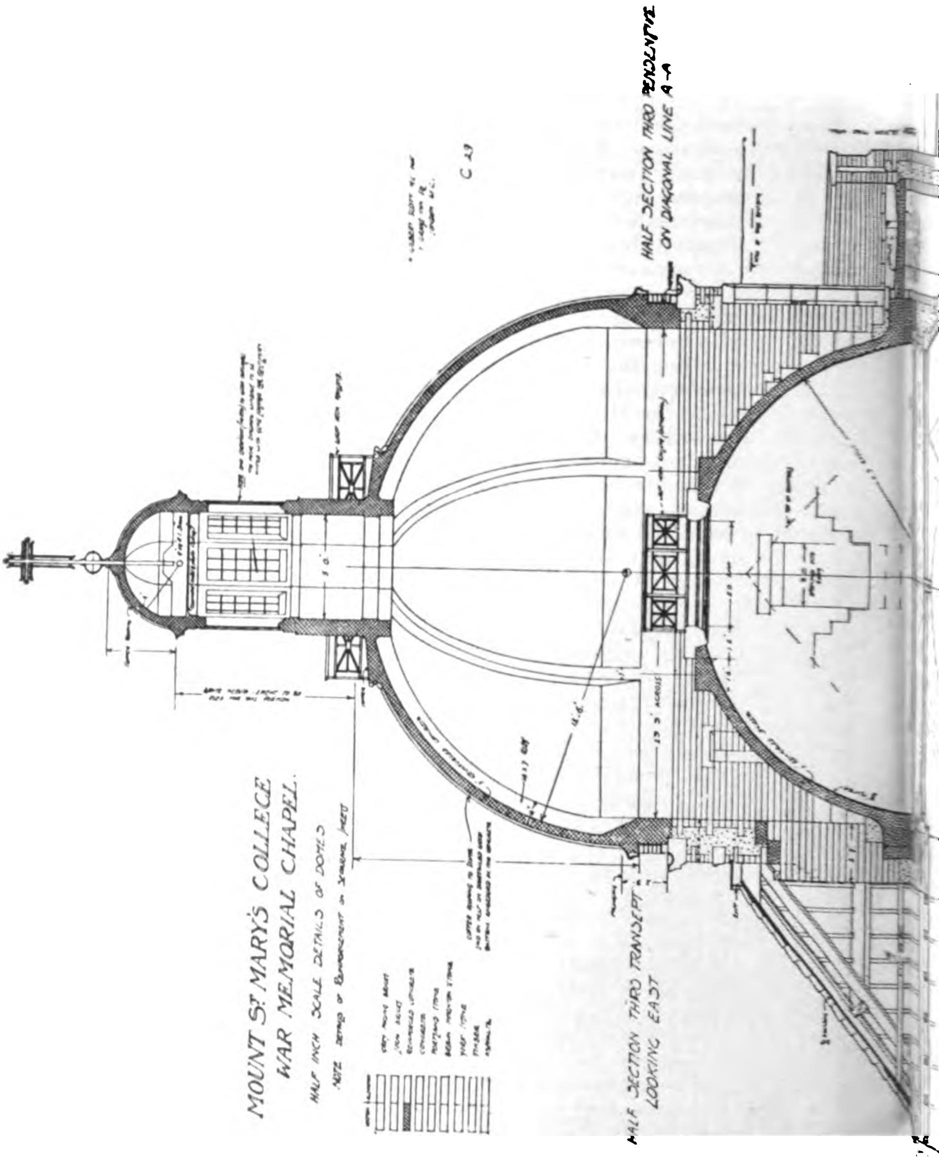
THIS is a second edition of this work, revised to meet changes in the standards of the American Steel Manufacturers. It should help to bridge the gulf between the technical college and practical work in the drawing office of a structural steelwork firm, and shows in a very practical manner how the detail drawings of beams, stancheons, bridge chords, and similar structural members should be prepared to correlate shop requirements with theory and erection facilities.

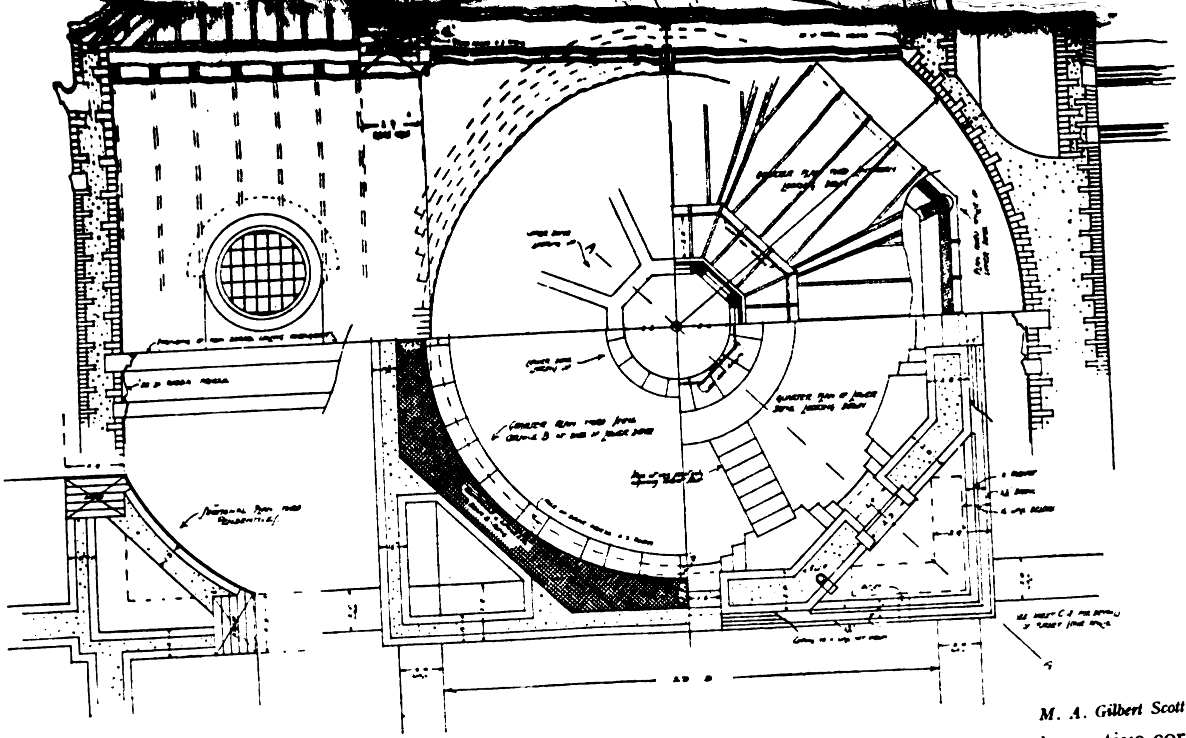
Sufficient theory is given to remind a student of some of the more elementary things which he should have absorbed at college, but the real value of the book lies more in showing how the practical requirements of the workshop are to be met.

The details and diagrams are very clear and good, and the book fairly represents good modern American practice, from which English practice only differs in minor details which need not worry a student. The book can be thoroughly recommended.—O. F.

Puzzolanic Cements.

A FIRST experiment with spent shale from the Scottish oilfields at the Department of Scientific and Industrial Research Building Research Board's Experimental Station, has, we are informed, produced a puzzolanic hydraulic cement of a tensile strength at three months' age greater than that required for Portland cement by the B.E.S.A. Standard Specification. The cement is of a light red colour.


MOUNT ST. MARY'S COLLEGE
WAR MEMORIAL CHAPEL.
HALF INCH SCALE DETAILS OF DOMES
NOTE. DETAILS OF REINFORCEMENT ON SEPARATE SHEET
HALF SECTION THRO PENDENTIVE
ON DIAGONAL LINE A-A
HALF SECTION THRO TRANSEPT
LOOKING EAST
C 23



M. A. Gilbert Scott, Architect.

[Reinforced concrete has been adopted entirely for the construction of the dome of this Chapel. The dome is carried on reinforced concrete beams. The pendentives, although they are not called upon to bear any load, are also constructed in the same material. The concrete in the outside face of the lantern will be left free from the centering, and the decorative cornices will be cast in moulds. The position of the lantern is such that it does not call for finely-finished details. The reinforced concrete work is being carried out on the "Kahn" system to the designs of the Trussed Concrete Steel Co., Ltd., 22, Cranley Gardens, South Kensington.]

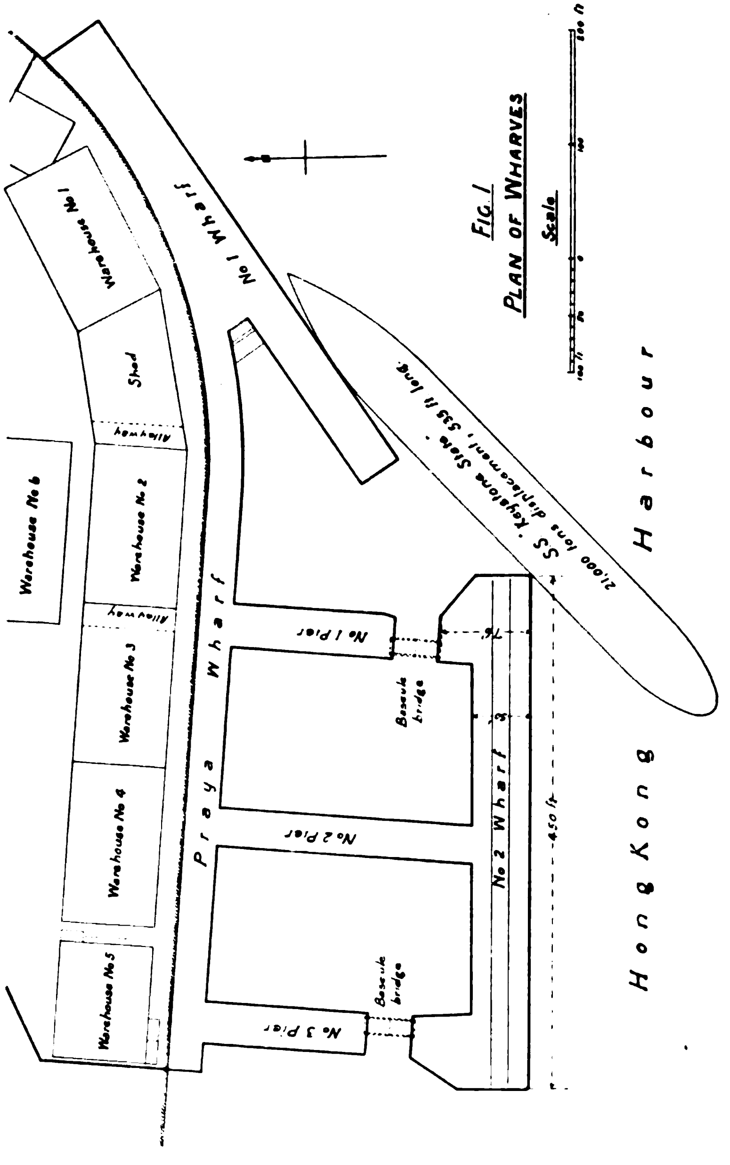


Fig. 1.—Reinforced Concrete Pile Wharf at Hong Kong. (See p. 535.)

REINFORCED CONCRETE PILE WHARF AT HONG KONG.

THE wharf illustrated herewith was constructed during the years 1916-18 at Hong Kong Harbour for Messrs. Alfred Holt & Co., to accommodate ocean-going steamers. The structure was designed by Mr. Somers H. Ellis, M.Inst.C.E., Engineer to the Company, and has several interesting features; the following particulars are taken from a paper read by Mr. Ellis before the Liverpool Engineering Society, and are reprinted from the *Transactions* of the Society by permission of the Council:—

DESIGN.

The general plan (*Fig.* 1) shows the lay-out. The face of the new wharf (No. 1) was aligned at an angle of 4½ deg. with the Praya wharf, instead of being parallel to it, in order to make a flatter angle with the face-line of the original steamship wharf (No. 1), and also to conform more nearly with the contour of the Harbour bottom. Existing pier No. 1 was shortened to 144 ft. in length, and pier No. 2 to 226 ft. A third (new) pier, 145 ft. long and 35 ft. wide, was built at the west side as the commencement of operations, together with a short extension of the Praya wharf. These subsidiary structures were of the same type as No. 2 main wharf.

No. 2 wharf is 450 ft. long and 50 ft. wide, with the addition of spurs at the back of each end, which leave entrances to the lighter basins enclosed by the piers. These entrances are spanned by two-leaf bascule bridges with overhead balance weights; they are raised and lowered easily by hand, and have proved very satisfactory. The wharf is thus served by three approaches, the centre one permanent, the outer two maintained almost constantly in commission while a ship is discharging. At other times the bridges are usually open, so as to permit of the free traffic of lighters and small craft in and out of the basins.

The question of stability then had to be considered. Borings taken on the site of the proposed wharf showed rock or hard decomposed granite at a reasonable depth throughout. Along the face-line this depth (below harbour bottom) varied irregularly from 18 ft. at the west end to 28 ft. at the east end, the overlying strata consisting of soft deposits which afforded little lateral support for piles. Above the rock was 1 ft. to 3 ft. of firm decomposed granite, then 5 ft. to 7 ft. of soft brown clay—evidently formed from the felspar constituent of the granite—and above that 10 or 12 ft. of sand mixed with grey silt, with a surface layer of soft mud. The depth of water along the wharf face at low tide varied from 35 ft. at the west end to 42 ft. on the east. Hence a pile-wharf, unless stiffened by underwater bracing or by other means, must necessarily have long piles with little lateral support between the bracing above low-water level and the hard decomposed granite where their points were embedded, 55 to 72 ft. below.

Owing to the necessity for preserving the lighter basin behind the wharf no support could be maintained by continuous piers, except in the centre, between the wharf and the shore; and the ends of the wharf had to depend for stability on the stiffness and dead-weight of the structure itself. These conditions were assisted by widening the wharf from its normal breadth of 50 ft. to 78 ft. at the east end, and 95 ft. at the west end, over a breadth of about 40 ft. in each case.

In the design as finally adopted, the wharf was supported on groups of three reinforced concrete piles, 14 in. by 14 in. in section, driven down to rock, and with their heads about 3 ft. 6 in. above low-water-level. The centres of the groups were 15 ft. apart across the wharf, and generally 20 ft. apart longitudinally. Over the groups of piles were lowered reinforced concrete cylinders, 4 in. thick and 5 ft. and 5 ft. outside diameter, in 6 ft. lengths. When in place, these were filled with mass concrete, so as to form above ground level a series of solid piers supported by piles from the rock beneath, and reinforced by the longitudinal steel in the piles, altogether twelve bars, 1⅛-in. diameter, to each pier. The heads of these piers were connected by a system of heavy reinforced concrete horizontal bracing, as shown in *Figs.* 2 and 4, the front and back longitudinal members being 2 ft. by 1 ft. 6 in. in section, the inter-

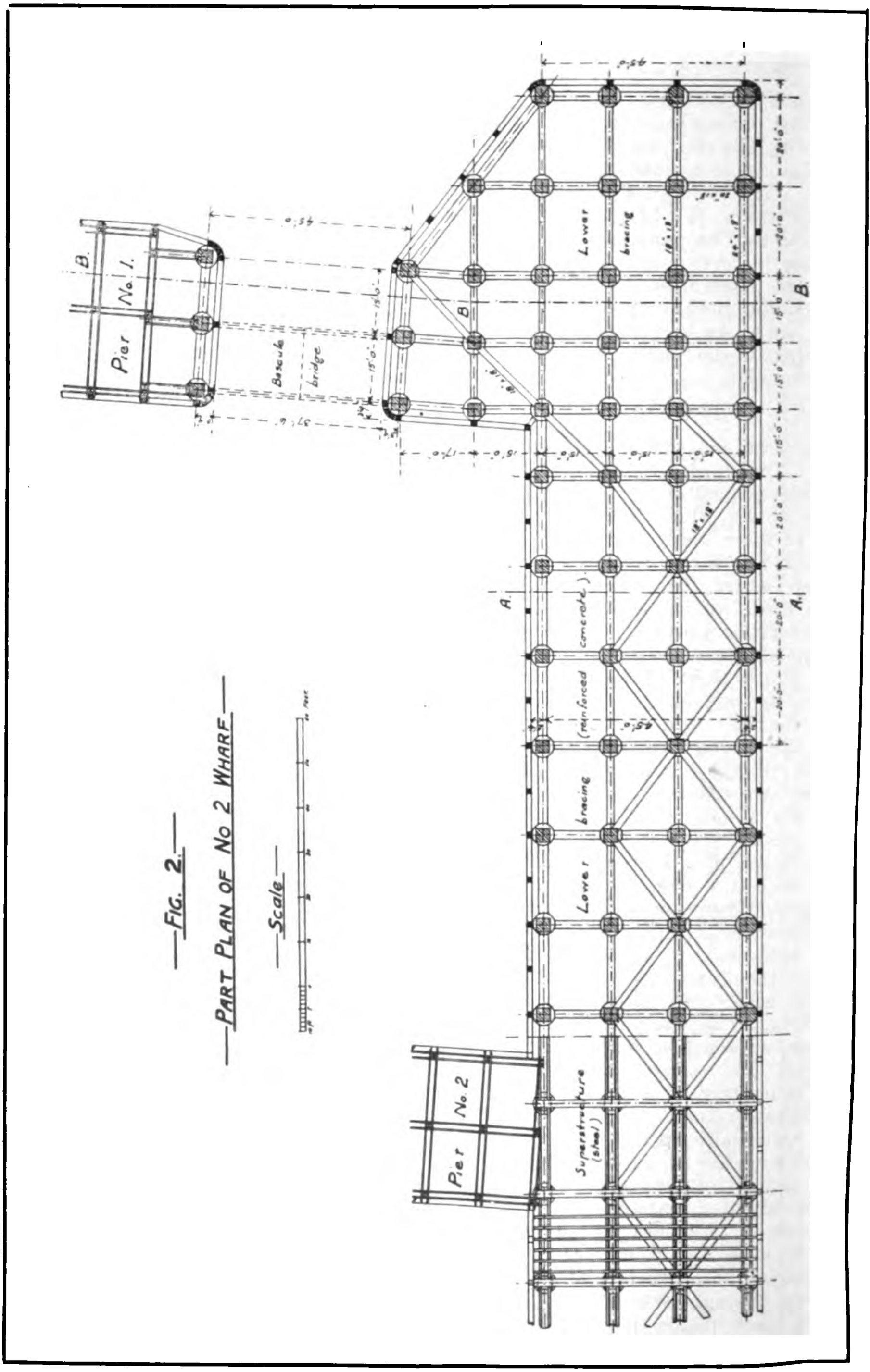


FIG. 2.

PART PLAN OF No 2 WHARF

FIG. 2.

mediate longitudinals 1 ft. 6 in. by 1 ft. 6 in., and the cross members 1 ft. 8 in. by 1 ft. 6 in. There was also a secondary system of 18 in. by 18 in. diagonal horizontal bracing immediately above this, as shown in *Fig.* 2. These diagonals, however, were designed merely as struts, and were dropped into place without any bond with the pier reinforcement. The bars of the transverse bracing, as will be noted from *Fig.* 4, overlapped within the area of the pier, and were interlaced with the heads of the pile-rods, thus forming a rigid horizontal framework just above low-water level.

Above the lower bracing the piers were formed of mass concrete, 3 ft. by 3 ft. in section, with a steel core consisting of a vertical 10-in. by 10-in. equal-flanged rolled steel joist, which penetrated 4 ft. 6 in. into the cylinder piers below, so as to form a connection between the piles and superstructure. The superstructure itself, apart from these concrete-encased piers, was of steel, with a timber plank deck on timber joists, all of Borneo hardwood. The diagonal and upper-cross bracing was formed of double 11-in. by 4-in. channels, the cross bracing being further strengthened by a plate riveted to the upper flanges, and the deck beams were double 18 in. by 7 in. steel joists. These members were all riveted, either direct or by means of cleats and gusset plates, to the vertical 10-in. by 10-in. joists, which formed the skeleton reinforcement of the piers above low-water-level. The wharf was designed as a framed structure, braced so as to resist overturning by the shock of a vessel coming against its face; and stability was assisted by the deposit of a mound of granite rubble, as shown in the cross sections (*Fig.* 3) after the structure was, except for the deck, completely erected.

For the sake of stability alone it might

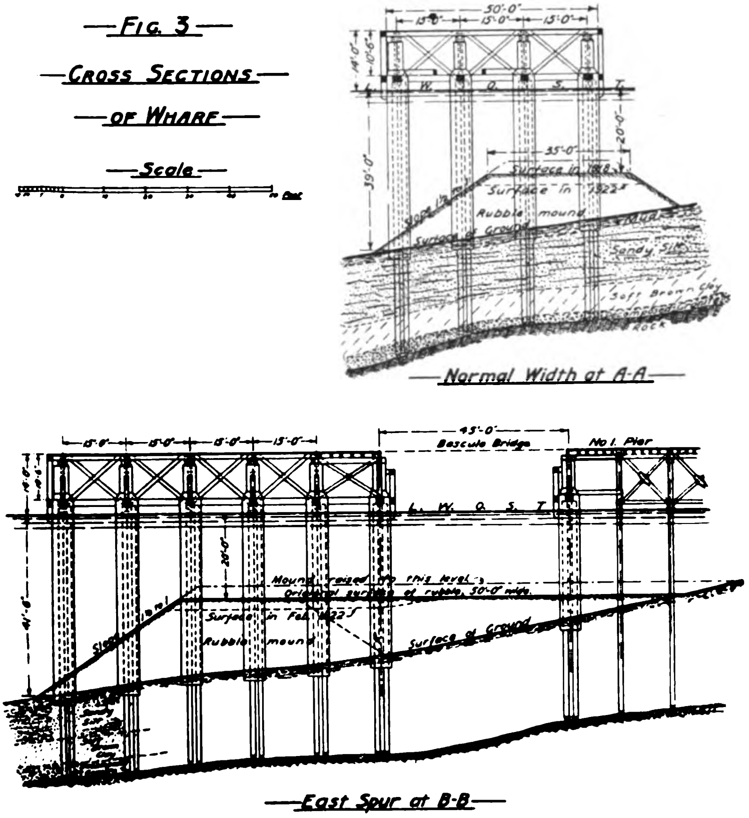


FIG. 3.

have been better to have designed the wharf of reinforced concrete throughout, with a heavy deck which would have assisted materially to that end. Questions of the permanence of the material had, however, to be considered. In a paper read before the Institution of Civil Engineers in 1914 he described the inadequate protective effect which 2 in. of concrete covering had afforded to the steel superstructure of No. 1 wharf on this work. He had furthermore examined a number of carefully-built structures of reinforced concrete in the neighbourhood of Hong Kong and the Straits Settlements, and had found so many instances of deterioration that he had come to the conclusion that cement concrete by itself was not an effective protection to steel for marine structures in tropical waters in the zone between mean-tide level and about 10 ft. above that level. So long as the concrete is wet every tide, a film of weed and other seagrowth appears to seal the pores of the concrete and prevent sufficient air

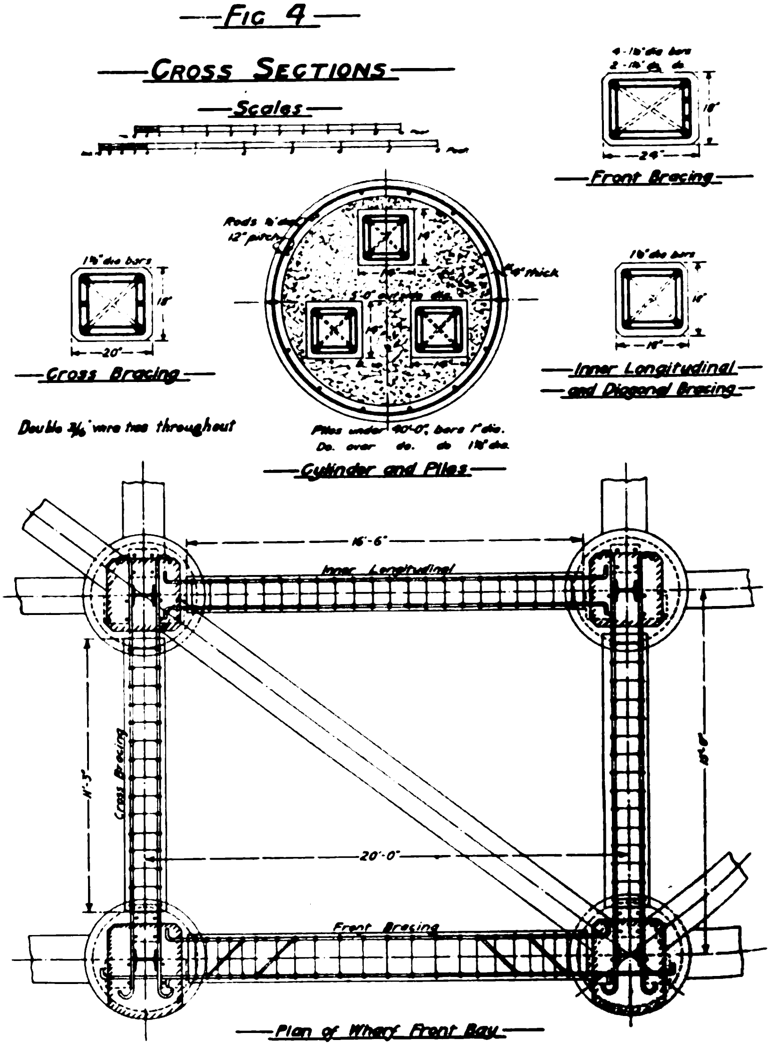


FIG. 4.

penetrating to the steel reinforcement to cause corrosion. Again, at a sufficient height above the sea, and particularly in a position freely exposed to sun and wind, good concrete appears to form an adequate and permanent protection even in such a climate as Hong Kong, where steel corrodes with extreme rapidity, and paint has to be continually renewed. Immediately above water level, however, extending down to high-water of neap-tides, and up to 6 ft. or more above high-water of springs and especially in such an enclosed situation as the underside of a wharf deck, the damp salt atmosphere has a tendency to penetrate even the most compact concrete, causing corrosion and enlargement of the steel reinforcement, consequent cracking of the concrete covering, and, unless measures of repair are promptly undertaken, more or less rapid deterioration of the structure. Subsequent experience has led the author to believe that this difficulty can be overcome by the use of a coating to the concrete—preferably one of an oily or tarry nature.

Before leaving the question of design, it may be pointed out that the diagonal horizontal bracing was arranged to spread the impact from a ship striking the face of the wharf, more especially on the narrow and unsupported portions on each side of Pier No. 2, over a greater length than that of the one or two bays immediately affected, and thus call into effect a greater resistance to withstand the shock. Had the diagonal bracing been continued for the full length of the wharf it would undoubtedly have added some strength, though on different lines, to the corner where the heaviest blow was actually experienced.

CONSTRUCTION.

Both piles and cylinders, as well as the lower bracing members, were moulded in a water-side yard a short distance away, carried by a travelling gantry on to a jetty built for the purpose, and conveyed on a pontoon to the site of the works. The piles were driven and cylinders lowered by a pile-driving frame and steam winch mounted on a pontoon, moored alongside a timber staging which was erected over the site of the wharf as it advanced. A concrete mixer installed on the outer end of No. 2 Pier supplied all concrete to be laid *in situ*, this being conveyed to place in trucks on trackways

—HALF CROSS SECTION OF No 2 WHARF—

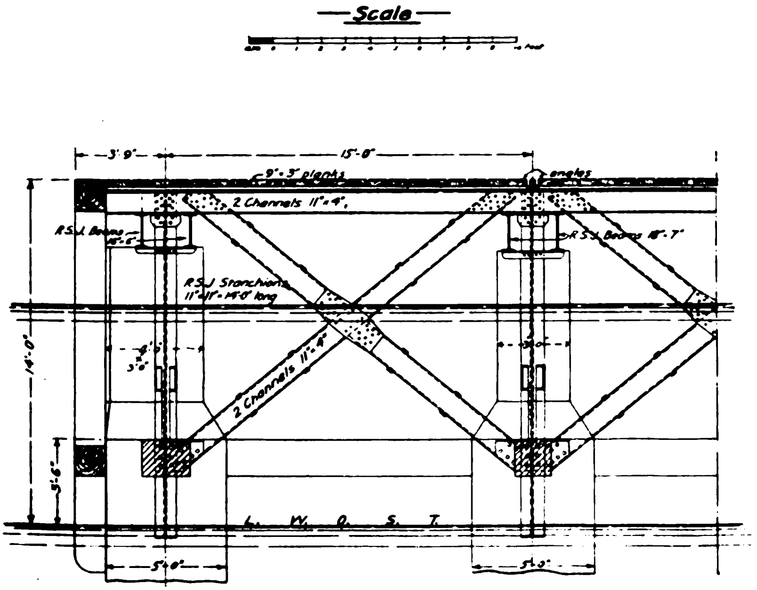


FIG. 5.

laid on the staging. The edges of the cylinders were moulded with rebates, so as to form a stepped joint, having an outside open groove between them. In the case of the first three or four cylinders for each pier these joints were made good with cement mortar above water level, and the set lowered together by wire ropes passing through steel rings projecting from the concrete of the bottom cylinder. In the case of succeeding sections, which were lowered singly, the outer circumferential grooves were filled with clay by a diver when the cylinders were in place. A level bearing was prepared by a diver in the sloping harbour bottom, and the cylinders guided into the correct position with the aid of templates adjusted to suit the piles, which, however, were generally very accurately placed and driven.

When the cylinders over one group of piles were in place up to about low-water level, a top section of variable length, notched to receive the lower bracing, had to be fitted so as to bring the lower pier head to the correct level. The practice was to mould in advance a considerable number of these "compensation" sections of the estimated lengths; but as it was impossible to foretell exactly how far the cylinders would sink into the mud—the usual allowance being 2 ft. 6 in.—some delay was occasionally caused by a special piece having to be moulded for the purpose as the work proceeded. The cylinders were then filled with concrete, mainly laid under water by means of small hopper buckets so shaped as to pass readily down the spaces between the piles and between the piles and cylinders. The concrete was gently worked into place by long bamboo rods from above, but otherwise not disturbed more than was necessary. An opportunity to test the quality of this concrete was afterwards afforded when one partially filled pier was forced over during construction by the action of a typhoon; and the concrete was found to be satisfactorily solid and homogeneous.

The top 15 ft. of each lower pier was pumped out and filled in with concrete in the dry.

The lower bracing, of reinforced concrete members pre-moulded on shore, was afterwards laid in place, the steel superstructure erected and riveted up and the square upper piers of mass concrete formed in place on top of the cylinders.

The whole of the concrete in the piles, piers and superstructure was gauged of 376 lb. Portland cement to 7½ cu. ft. of sand and 15 cu. ft. of granite broken to pass a ¾-in. ring—being a mixture rather stronger than 1, 2, 4—with the exception of the cylinder filling laid in the dry, which had 9½ cu. ft. of sand and 19 cu. ft. of stone to 376 lb. of cement.

In repairing some damage done to the structure, during construction, by a typhoon, it was found that the original concrete filling deposited under water had penetrated in a solid, homogeneous mass to about 18 in. below the surface of the outer mud, about 1 ft. above the bottom edge of the cylinder; and, so far as it was cut into, evidence was obtained that a satisfactory hearting had been provided.

General Post Office, Singapore.

THE design of Messrs. Edmond Coignet, Ltd., of 125, Gower Street, W.C., has been selected for carrying out the new General Post Office at Singapore in reinforced concrete, for the Crown Agents of the Colonies. The building is to be erected on a triangular site, and will be 120 ft. in height (six stories). The principal façade will be 400 ft. long, and the other two façades 300 ft. long; it is to be built on a reinforced concrete raft foundation, and will have a flat roof. The intermediate courtyards will be covered. The reinforced concrete columns, etc., on the elevations will be cased with granite. The architect is Major P. H. Keys, D.S.O., F.R.I.B.A., of Singapore.

A Tall Concrete Chimney.

A CHIMNEY being constructed at the briquetting factory of the State Electricity Commissioners, Yalloum, Victoria, will be 332 ft. high. The diameter of the concrete base is 45 ft. The barrel of the chimney, as well as its base, will be in reinforced concrete, 13½ ft. diameter, measured inside the concrete, and 11½ ft. diameter inside the brick lining.

"CONCRETE ROADS."

THE large number of advance orders received in response to the announcement in our last issue of the publication of a second edition of "Concrete Roads" is evidence that the necessity for improved methods of road construction to withstand the wear and tear of the fast and heavy traffic of the present day is fully realised and that there is a considerable demand for information on the all-concrete road.

In 1920 we published the first edition of our book entitled "Concrete Roads," for which there has been a steady sale. The first edition is now exhausted, and we have therefore published a second edition, which has been brought right up-to-date to include current practice.

The chapters include: Concrete Roads in England, Scotland, Wales, and Ireland, with latest Official Reports as to their condition, Weight of Traffic, Maintenance Cost, etc.; Concrete Roads in Industrial Works; Concrete Roads in Australia, New Zealand, Canada, and America, with latest reports; The International Road Congress at Seville, May, 1923—Extracts from Reports and the Resolutions; The Bates Tests of Concrete and other Roads; Mechanical Appliances used in the Construction of Concrete Roads; Concrete Kerbs and Channelling; A suggested form of Specification for the making of Concrete Roads; Pavements, Fence Posts, Telegraph and Lamp Posts, etc.; Useful Hints and Data for making Concrete and the Choice of Aggregates; Press and other Opinions on Concrete Roads; etc. In the compilation of the book we have had the assistance of engineers who have had a great deal of practical experience in laying concrete roads, and whose work is amongst the most successful in the country.

The book is well illustrated throughout, and the particulars of the methods of construction used by different Borough Surveyors should be particularly valuable to those about to lay new roads—the volume is, indeed, the only publication in which this information has been collected together in a convenient form. The book is now ready (price 5*s.* net), and may be obtained from the Trade Manager, Concrete Publications, Ltd., No. 4, Catherine Street, Aldwych, London, W.C.1, by filling in and posting the form on p. xxvii.

A correspondent in California writes: "I have driven through all the Western States, on deserts and mountains, in all kinds of weather, and in my opinion the concrete road has no rival. In California we are building concrete roads in every direction, and a new suburb has just been opened that possesses 60-ft. concrete roads throughout. The smallest county of this State—Orange County—has nearly 300 miles of concrete highway, not reckoning the roads of the incorporated cities. A 2,000-mile concrete road along the Pacific Coast from Canada to Mexico will be completed in a few months, and from travelling over part of the finished road I can assure you it is the most delightful auto trip I have made. After many experiments it has been decided that 8-in. and 10-in. is the best thickness for the concrete surface. This State (California) expends enormous sums annually on its roads, and over 1,000,000 cars have been registered in California so far in 1923. Concrete roads are safer, smoother, more economical, and far superior to any other kind." Such a letter from a road user can hardly be imagined in this country; it suggests, however, that with the increasing adoption of the concrete road in this country, there will be but little to complain of on the score of bad roads.

ESTIMATING FOR CONCRETE WORK.

In a paper read recently before the American Concrete Institute, Mr. Frank R. Walker dealt with the subject of estimating for the various items in concrete construction.

If estimates were to be accurate, he said, so that the contractor would feel safe in taking work on his own figures, the various branches of work should be estimated in exactly the same manner in which they were erected in place in the structure, using each completed operation as the basis of measurement and pricing the work. Regardless of the method used in preparing the estimate, if the contractor's field organisation was incompetent or inefficient the best prepared estimate would not make the contractors' business a profitable one.

Each class of concrete work should be estimated separately, such as footings and foundation walls, column footings, retaining walls, reinforced concrete columns, beams and girders, stairs, structural floor slabs, etc., because each was a separate and distinct class of work, involving special details of construction not found in the others, and subject to considerable variation in the cost of the completed units.

When estimating the cost of concrete for foundations and retaining walls, the cost of the wood or metal forms, reinforcing steel, and the cost of the concrete itself, should each be estimated separately, as they were all separate kinds of work and in many instances installed by different trades.

FORMWORK.

Formwork should always be estimated by the number of sq. ft. of forms required for any given piece of work. Light walls would require only 2 or 2½ ft. of lumber, b.m., to construct 1 sq. ft. of forms complete, while heavy retaining walls would require 3½ to 4 ft. of lumber per sq. ft. of forms. Naturally, the more lumber required per sq. ft. of forms the higher the material and labour costs would be.

It was not a difficult matter to strike an average of the amount of lumber required to construct 1 sq. ft. of forms for various types of walls, and by so doing valuable costs both by the sq. ft. of forms and per thousand ft. of lumber were furnished. That was especially valuable when comparing costs on various jobs. The cost of removing the forms should also be kept separate from the cost of framing and erecting, but the same unit of measurement should be used throughout.

REINFORCEMENT.

Reinforcing steel should be estimated by the pound or ton. The labour for bending the steel should be kept separate from the cost of placing the steel in the structure ready to receive concrete.

COLUMNS.

The forms for square concrete columns should be estimated by the sq. ft., taking the entire girth of the column and multiplying by the height. For a column 17 in. sq. and 12 ft. high, forms would be required on all four sides of the column, making a total girth of 72 in. or 6 ft., which, multiplied by the height, would give 72 sq. ft. of forms required for each column. Where 2 in. lumber was used it ordinarily required about 2½ ft. of lumber to construct 1 sq. ft. of forms, including bracing, clamps, etc. In order to prepare accurate estimates on the cost of such work it was advisable to estimate the cost of framing and erecting the column forms separate from the labour cost of removing them after the concrete had been placed. The same units of measurement should be used in both instances.

The forms for octagonal concrete columns presented a somewhat different problem in estimating. Regardless of the diameter of the column, there were eight different sections to be formed, and it did not cost any more to frame an 8-in. section than a 6-in. one, so it was a question whether it was not advisable to estimate the labour cost at a certain price per column, making a slight allowance for the difference in size. However, regardless of the diameter of the column, the labour cost of framing and erecting the forms in place should be kept separate from the labour cost of removing the forms after the concrete had been placed.

Metal moulds were ordinarily used for forming round concrete columns. The column moulds were fabricated in the shop and delivered to the job in sections, so that a certain price should be allowed for the erection and removal of each column mould, stating the diameter and height of the column, and whether a plain shaft or one having a flared head, as the latter cost considerably more to erect than the former. The cost of the erection of the forms should be kept separate from the removal of them, if possible.

The column reinforcing steel should be estimated by the pound or ton. Inasmuch as the spirals were usually furnished fabricated and the steel cut to length, the labour estimate should include assembling, wiring, and placing, based on the actual weight of the steel used.

The concrete in the columns should be estimated by the cu. ft. or cu. yd.; the materials being estimated according to the proportions of cement, sand and gravel used, and the labour costs of mixing and placing should be estimated separately.

BEAMS, GIRDERS AND LINTELS.

There was a diversity of opinion among contractors regarding the correct method of estimating forms for concrete beams, girders, and lintels. When used in connection with concrete floor slabs, the depth of the beam or girder ordinarily included the thickness or depth of the slab, so when estimating the quantities and cost of beam and girder forms, he thought the proper method was to take that portion of the beam or girder that was exposed beneath the slab, while some contractors estimated forms for the full depth of the beam. For instance, if a concrete beam were 16 in. by 24 in. in size and the thickness of the slab 6 in., the actual depth of that portion of the beam requiring forms was 18 in., the other 6 in. being made up in the thickness of the concrete floor slab. Where beams and girders were used around stair-well openings, elevator-shafts, door and window lintels, etc., where one side of the beam was exposed for the full depth, then the girth of the beam would be 18 in. plus 16 in. plus 24 in., or a total of 58 in.

Beams might be constructed of either 1 in. or 2-in. lumber or a combination of the two, but the labour cost per sq. ft. remained practically the same in either instance. The 1-in. lumber was somewhat easier to frame, but it required more bracing than the 2-in. lumber, so that the advantages of one practically offset the other. When the estimated labour cost should be by the sq. ft. of forms actually constructed, the labour cost of framing and erecting the forms should be kept separate from the labour cost of removing the forms after the concrete had been placed.

The reinforcing steel should be estimated by the pound or ton, based on the actual amount of steel required. Bending and fabricating should be kept separate from the labour cost of placing the steel in the forms ready to receive the concrete.

REINFORCED CONCRETE FLOORS.

There were so many types of reinforced concrete floors being used to-day that it was almost necessary to use a different method when estimating each type. The sq. ft. method of measuring the forms could be used in all of them, but there were so many special conditions entering into the construction of the forms for the various types of floors that each must be given careful consideration when preparing the estimate. The amount of lumber required to construct 1 sq. ft. of forms for reinforced concrete floors would vary from about 2½ to 4½, which included sheathing, joists, stringers, uprights, sills, wedges, bracing, etc. That variation was due to the height of the ceiling, and for that reason it was well to check the sq. ft. method with the price per 1,000 ft. of lumber before pricing the estimate.

It was not necessary to figure out the exact quantity of lumber required in the construction of the slab forms for each job estimated, but a table could be easily prepared showing the approximate amount of lumber required to construct 1 sq. ft. of forms for different slab thicknesses and varying ceiling heights.

Before deciding upon the material cost per sq. ft. of forms, it was advisable to take into consideration the number of times each floor of forms might be used in the construction of the job. For instance, if it required 3 ft. of lumber to construct 1 sq. ft. of forms, with lumber at 2½d. per ft. the lumber cost 1 sq. ft. of forms would be 7½d. If the lumber were used two or three times in the construction of the job, that cost might be divided by 2 or 3 to obtain the lumber cost per sq. ft. of forms. To that a small allowance should be added for breakage, waste, nails, etc.

SOLID CONCRETE AND SPECIAL FLOORS.

Where solid concrete floors were used in connection with beams and girders (either of reinforced concrete or structural steel, fireproofed with concrete), the exact floor area between beams and girders should be obtained, and should not include the width of the beams and girders, as those forms had previously been taken care of under beams and girders, and all duplication of quantities should be avoided if accurate estimates were to be obtained.

The same method should be used when estimating floors of flat-slab construction except that the area of the depressions at the column heads should be taken off separately, as considerable extra labour was involved in framing out for those depressions. The entire area of the floor might be estimated at a certain price per sq. ft., and then an additional allowance might be made to take care of the additional labour required for framing of depressions in slabs.

Other types of floor construction where concrete was used in conjunction with tile, etc., usually required the lightest type of temporary framing for forms on account of the small weight of the floor slab itself. In all instances the exact floor area should be obtained and the work priced by the sq. ft. of forms. On work of that kind some contractors covered the entire floor with wood sheathing while others placed only 2-in. by 6-in. or 2-in. by 8-in. planks 12, 16 or 20 in. on centres to support the ends of the tile or domes and form the bottom or soffit for the concrete joist which was to be placed.

It was certainly advisable to keep separate costs on the various types of floor construction both by the sq. ft. of form and by the 1,000 ft. of lumber, as that helped materially in preparing more accurate estimates, permitting a comparison of costs by both methods.

LABOUR IN RELATION TO VOLUME OF CONCRETE.

The actual concrete on all of the various types of floors should be estimated by the cu. ft. or cu. yd., but there was a vast difference in the labour cost of placing concrete on a job having 10-in. or 12-in. floor slabs than those using clay tile or metal domes, having concrete joists 4 in. or 5 in. wide with 2 in. or 2½ in. of concrete placed over the top of the tile. The former would require 4 or 5 times as much concrete per sq. ft. of floor as the latter.

On heavy slabs it was possible to place a large amount of concrete in a comparatively small floor area, while with the combination types of floor it was necessary to cover a large floor area and place a comparatively small amount of concrete. Floors of that type required considerable extra labour placing the concrete in small joists, using care to see that all reinforcing steel was covered, as well as extra labour in spreading and grading the concrete for the thin top slab. While the cu. ft. or cu. yd. method of measurement might be used for either type floor, the labour cost on one would be much higher than on the other.

Where the floors were to have a cement-finish top, the top should be estimated separately in all instances. The thickness of the top might vary from ½ in. to ¾ in. in thickness, and was composed of cement and sand, an entirely different aggregate than the mass concrete. That would involve separate material and labour costs, such as cement finishers and helpers, and placing screed.

CONCRETE STAIRS.

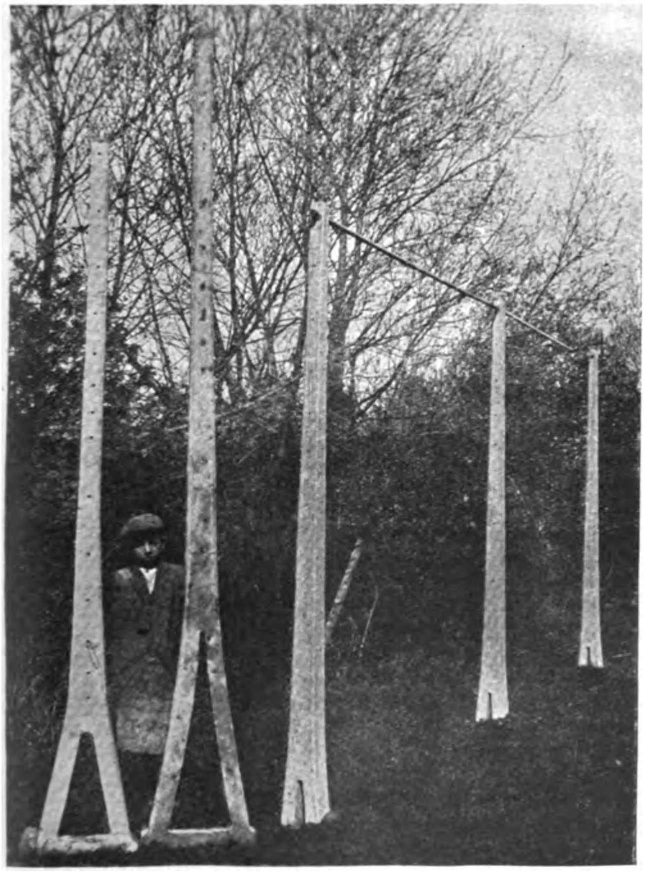
Nearly every contractor had his own method of estimating concrete stairs. Many took the underneath surface or soffit of the stairs to obtain the forms required, while others took the area of the soffit plus the number of lineal ft. of risers. It was a difficult proposition and one that was not easily checked.

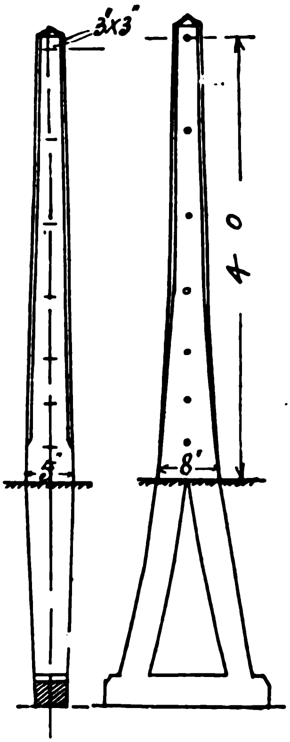
He had used the following method of estimating concrete stairs for a number of years, and it had worked out very well: He first took the length of the stringers of each flight of stairs, making a note of the number of treads in each. According to his experience on the job, the most time was required in laying-out the stairs, cutting the stringers and placing them ready to receive the sheathing and risers. He usually allowed a certain number of hours carpenter time to lay-out, cut, and erect the stringers. He then estimated the sheathing in the same manner as was used in estimating slab forms, making an allowance in the price. He then mentioned the number of treads and the length of them, together with the number and length of the risers to be placed. In making up the estimate in that manner, the estimated and actual costs compared very favourably.

The concrete was estimated in the usual manner to obtain a number of cu. yds. of concrete to place. If the stairs were to have a cement finish, he usually allowed 18 in. to take care of the width of each tread and riser and multiplied that by the length and the number of treads to obtain the number of sq. ft. of finish.

When estimating the quantities of mesh or fabric reinforcing, he thought the actual area of the surface to be covered should be obtained and then a certain allowance made to take care of the laps, the same as was used when figuring wood sheathing or flooring.

In his opinion the item of plant and equipment should be estimated separately, allowing a certain price for concrete mixers, hoisting engines, hoisting towers, and other equipment, as in many instances that cost would be the same whether the job contained 1,000 or 5,000 cu. yds. of concrete, so that if that was estimated separately it was much easier to obtain costs from the work that were authentic and of value when preparing future estimating.





Specially strong post for taking strains at corners or on curves. Holed both ways for plain wire or netting fence.

CONCRETE FENCE POST.

CONCRETE POSTS FOR ROSE ARCHES OR TENNIS COURTS.

[The above illustrations show some concrete posts manufactured by the Wharf Lane Concrete Co., Ltd., of Ilminster, Somerset. The posts for rose arches are 8 ft. and 9 ft. 6 in. in height (above ground level). The post for curves and corners in fencing is 1 cwt. 1 qr. in weight.]

THE BRITISH STANDARD SPECIFICATION FOR PORTLAND CEMENT.

At the last meeting of the Institution of Structural Engineers, Mr. D. B. Butler, A.M.Inst.C.E., F.C.S., read a paper in which he compared the British Standard Specification for Portland Cement with other specifications, and urged that the British Specification should be revised and strengthened.

In the course of his remarks he said :

The British Standard Specification for Portland cement was first issued in 1904 by the Engineering Standards Committee (now the British Engineering Standards Association).

Although the provisions of the first British Standard Specification were open to some criticism, and did not, of course, meet with the approval of everyone interested, its appearance gave general satisfaction, both to users and manufacturers ; to the former, because they had a specification ready prepared for their use, and were saved the trouble and responsibility of framing one of their own ; and to the latter because they henceforth had only one specification to fulfil, instead of possibly a dozen.

Revised editions of the British Standard Specification were issued from time to time, as necessity arose, the tendency naturally being to strengthen and stiffen the conditions, so that the last edition of 1920 is very considerably stiffer than that of the first edition of 1904 ; as will be seen later, the 1920 edition again requires revision and strengthening, in view of the improvements in modern cement-making practice.

Take, for instance, the matter of fineness of grinding ; in the 1904 edition 22½ per cent. residue was allowed on a 180 sieve, and 3 per cent. on a 76 sieve ; the former was reduced in 1907 to 18 per cent., and in 1915 to 14 per cent., at which figure it now stands in the current 1920 edition. In view of present-day cement mill practice, this is far too high ; 8 or 10 per cent. would be a more reasonable figure, having regard to the fact that most British cement for export purposes is ground down as low as 5 per cent. The latter is probably on account of foreign competition, since judging from samples of cement from other countries, which the author is continually having through his hands for analysis and report, the British Standard Specification conditions, as regards fineness, are rather lightly esteemed by foreign and Continental manufacturers.

Table I gives the fineness of 25 foreign samples, received during the past three or four months, not in any way selected. It will be observed that there is one very coarse Belgian sample of 19 per cent. on a 180 sieve ; otherwise the maximum is 10·5 and the minimum 1·6, the mean of the whole 25 being 7·4 per cent.

Table II gives examples of cement from ordinary English contracts, sent the author for examination and report, which bear out the suggestion that the conditions of the British Standard Specification as regards fineness are altogether too lenient. These likewise are 25 consecutive examples, and are not in any way selected.

It must not be forgotten, however, that finer grinding means extra cost to the manufacturer, since to obtain the extra fineness he has to reduce the feed, and consequently the output of his grinding mills ; to maintain the same output of finer cement, therefore, involves the installation of extra grinding plant. On the other hand, the finer a cement is ground the greater is its cementitious value, or power of cementing together particles of sand and other inert material ; although they have a slight value, inversely proportionate to their diameter, the coarser particles of clinker are more or less of the nature of a diluent.

One test, which was included in the earlier editions of the British Standard Specification, and has been very properly dropped in the 1920 issue, is that of specific gravity. To all intents and purposes the specific gravity of an ordinary Portland cement is governed entirely by the facilities it has been afforded of absorbing water and carbonic anhydride from the atmosphere, etc., since burning. Specific gravity was originally supposed to indicate the degree of calcination to which the clinker had been subjected in the course of manufacture, a high specific gravity indicating a well-burnt, and a low specific gravity an under-burnt, material ; the author's researches, however, showed conclusively that when freshly-burned the specific gravity of the two materials was identical, and it was only after more or less exposure to atmospheric influences that any difference took place.

Another reason adduced for determining specific gravity was to detect adulteration with slag, limestone, etc., but since these materials can be more effectively detected by means of the microscope, or by the chemical analysis prescribed by the British Standard Specification, the determination of the specific gravity of a cement is, in most cases, sheer waste of time.

Referring next to the tests for strength, 7 and 28 days' neat and sand tensile tests only have so far been included in the British Standard Specification, in spite of the German and more general Continental practice of relying entirely upon resistance to crushing stress. There is doubtless much to be said for crushing tests as the more usual stress to which concrete is subjected in use, but tensile tests are more convenient and less expensive to carry out, and in the author's opinion are quite as good an indication of the strength of a cement as crushing tests.

In the 1904 edition the minimum 7 days' neat test was fixed at 400 lb. per square inch, and the sand test at 150 lb., with a proportionate increase in each case after 28 days; these figures held good till 1914, when the minimum was raised to 450 and 200 lb. respectively, which remained unaltered in the last 1920 edition. Judging from the samples from various sources which pass through the author's hands daily for testing purposes, these figures are far too low, and very inadequately represent the average strength of modern Portland cement, either British or foreign. The best Portland cement at the present time develops a tensile strength of nearly double that specified by the British Standard Specification, while the average

TABLE No. I.

FINENESS OF GRINDING OF FOREIGN PORTLAND CEMENTS.

No.	Origin.	Residue on Sieves Nos.	
		180 x 180 %	76 x 76 %
1	Belgium	9·5	1·2
2.	Sweden	4·0	nil
3.	Belgium	19·0	1·5
4.	Sweden	4·5	0·1
5.	India	6·9	nil
6.	Belgium	10·5	0·2
7.	Poland	9·0	0·1
8.	France	7·7	trace
9.	Sweden	3·6	,,
10.	Esthonia	3·9	nil
11.	Sweden	4·2	0·1
12.	Germany	5·4	0·2
13.	Denmark	10·0	0·2
14.	Denmark	9·0	0·3
15.	Holland	8·2	0·5
16.	Poland	4·6	nil
17.	Denmark	10·4	,,
18.	Spain	6·0	trace
19.	Belgium	6·3	nil
20.	Belgium	7·0	,,
21.	Denmark	1·6	,,
22.	Belgium	7·9	0·2
23.	Egypt	8·4	0·3
24.	Belgium	8·2	0·1
25.	Dalmatia	8·8	0·6

TABLE No. II.

FINENESS OF GRINDING OF ORDINARY COMMERCIAL BRITISH PORTLAND CEMENTS.

No.	Residue on Sieves Nos.	
	180 x 180 %	76 x 76 %
1.	5·1	nil
2.	5·6	0·2
3.	2·0	nil
4.	6·8	0·2
5.	9·7	0·5
6.	1·8	trace
7.	7·7	0·5
8.	7·0	0·3
9.	5·7	0·3
10.	5·3	nil
11.	6·2	0·1
12.	6·2	trace
13.	10·0	0·3
14.	4·2	0·1
15.	8·2	trace
16.	7·2	0·2
17.	6·5	trace
18.	5·0	,,
19.	3·3	nil
20.	6·8	,,
21.	9·3	0·3
22.	4·3	nil
23.	9·2	0·3
24.	3·9	nil
25.	11·7	0·3

is well over 50 per cent. higher; in Table III are given the 7 days' neat and sand tests of 25 consecutive samples, received by the author during May, 1923, for examination, which illustrates the above point.

In the author's opinion, therefore, the 7 days' tests might be raised to 600 lb. neat and 300 lb. sand, without causing any anxiety whatever to the average manufacturer; in fact, when asked by clients to express an opinion as to the strength of a sample, he has taken the foregoing figures as a basis, reporting anything below 600 and 300 lb. respectively as below the average of good Portland cement, and figures above that strength as above, or considerably above, the average as the case may be.

The majority of samples received from abroad, and more especially from Scandinavia, develop a somewhat higher strength, on the average, than those shown in the foregoing table, and the same remark applies practically to the whole of modern cement making practice; samples received from France, Belgium, Germany, Spain, Italy, the Baltic States, and even so far afield as China and Japan, give results so far in excess of the British Standard Specification requirements as to suggest to foreign countries that a low standard is accepted in this country. This, in the author's opinion, cannot be too strongly deprecated, and he would like the British Standard to be looked upon as the best in cement as in other manufactures.

When discussing the comparative strength of English and foreign cements a point that may be considered usefully is the Standard Specification adopted in other countries, and the method of carrying out the tests. The specifications with which the author has most frequently been brought into contact, with regard to testing shipments for export purposes, have been those of the U.S.A. and of the South American States, principally that of the Argentine Republic. The latter is based largely on German practice, and the German type of mould of 5 sq. c.m. sectional area is used for tensile tests. The requirements of the Government specification of the U.S.A. and of the Argentine Republic compared with the British Standard Specification are given in tabular form in Table IV, the metric units of the Argentine specification being converted into English weights and measures for more convenient reference.

The methods specified, however, of carrying out the tests are so widely different from those of the British Standard Specification that the figures are by no means comparative, unless these different methods are taken into account. The author carried out the usual physical tests with the same cement, in accordance with three specifications. For this purpose four different cements, each representing a particular class of material, were chosen from samples submitted for testing in the usual course, as under:—

(A) Belgian natural cement.
(B) English rotary kiln cement.
(C) English chamber kiln cement.
(D) "Ciment Electrique."

The results of the comparative tests were as follows, the preliminary tests for fineness, setting time and soundness being given in Table V and the tests for tensile strength in Table VI.

Referring first to the neat tests, the

TABLE No. III.

7 DAYS NEAT AND SAND TESTS OF TWENTY-FIVE CONSECUTIVE SAMPLES RECEIVED MAY, 1923.

No.	Origin.	7 Days Neat.	7 Days Sand.
1.	British.	796 lbs.	412 lbs
2.	British?	788 „	432 „
3.	British.	757 „	332 „
4.		648 „	279 „
5.	Danish.	838 „	366 „
6.	British.	768 „	349 „
7.	„	747 „	454 „
8.	Danish.	844 „	444 „
9.	British.	764 „	386 „
10.	German.	807 „	394 „
11.	British?	790 „	420 „
12.	British.	695 „	245 „
13.	Egyptian.	685 „	339 „
14.	British.	816 „	429 „
15.	Swedish.	746 „	392 „
16.	Belgian.	777 „	316 „
17.	British.	799 „	454 „
18.	„	856 „	396 „
19.	British?	498 „	246 „
20.	British.	783 „	398 „
21.	„	907 „	423 „
22.	„	562 „	352 „
23.	Swedish.	680 „	415 „
24.	British?	585 „	335 „
25.	British.	839 „	482 „

current U.S.A. specification, as will be seen on reference to Table IV, calls for sand tests only, and, therefore, in order to make the comparative series complete the method of preparing neat briquettes specified in the previous 1912 edition of the U.S.A. Specification was adopted. This method involved the use of a considerably wetter mixture than the British Standard Specification method, as will be observed on reference to the table. According to the 1912 U.S.A. Specification the correct amount of water is used when a ball composed of the neat cement paste, about 2 in. in diameter, dropped from a height of 2 ft. upon a hard, smooth surface, is not dry enough to crack and is not wet enough to flatten more than half of its original diameter. In filling the moulds the paste is pressed in firmly with the fingers and smoothed off with a trowel, without mechanical ramming. This wetter mixture and method of filling the moulds results in a considerably lower

TABLE No. IV.

BRITISH STANDARD SPECIFICATION, U.S.A. GOVERNMENT SPECIFICATION, AND ARGENTINE GOVERNMENT SPECIFICATION REQUIREMENTS SUMMARIZED IN TABULAR FORM.

Specification.			British Standard	U.S.A.	Argentine.
FINENESS OF GRINDING % Residue on	200 x 200 Sieve		—	Max. 20%	—
	180 x 180 ,,		Max. 14%	—	Max. 30%
	76 x 76 ,,		,, 1%	—	,, 5%
SETTING TIME	Vicat Needle	Initial	Min. 20 mins.	Min. 45 mins.	Min. 35 mins.
		Final	Max. 10 hrs.	Max. 10 hours	Min. 3 hours Max. 10 ,,
	Gilmore Needle	Initial	—	Min. 60 mins.	—
		Final	—	Max. 10 hours	—
SOUNDNESS	Le Chatelier Test	After 24 hrs. aeration	Max. 10 mm.	—	Max. 10 mm.
		or ,, 7 days ,,	,, 5 ,,	—	—
	Boiling Steam Pat		—	Sound	—
	Faija Warm Water Pat		—	—	Sound
	Cold Water Pat		—	—	Sound
	Damp Air Pat		—	—	Sound
SPECIFIC GRAVITY			—	3·10 Optional	3·05
TENSILE STRENGTH in lbs. per sq. in.	Neat Cement ...	7 days	450	—	427
		28 ,,	538	—	469 or if 7 days = 570 nothing inferior
	3 1 Sand Cement Mortar ...	7 ,,	200	200	170
		28 ,,	250	300	227 or if 7 days = 227 at least 241

TABLE No. V

Tests for Fineness, Setting Time, and Soundness of Four different Cements under British Standard Specification, U.S.A. Government Specification, and Argentine Government Specification conditions.

SAMPLE.	FINENESS OF GRINDING.					SETTING PROPERTIES.								SOUNDNESS					
	B.S.S.		U.S.A	Argentine.		B.S.S		U.S.A				Argentine		B.S.S	U.S.A	Argentine.			
	% Residue on Sieves Nos.					Vicat Needle.		Vicat Needle		Gilmore Needle.		Vicat Needle.		Chatelier Test. 24 hours aeration.	Boiling Steam.	Cold Water.	Damp Air	Faija Warm Water.	Chatelier Test 24 hours aeration
	180×180	76×76	200×200	180×180	76×76	Initial	Final	Initial	Final	Initial	Final	Initial	Final	Expansion.	Pat	Pat.	Pat.	Pat	Expansion
A. BELGIAN NATURAL CEMENT	11·9	1·1	20·0	11·9	1 1	3¼ hrs.	6 hrs.	4⅜ hrs.	8 hrs.	6 hrs.	9½ hrs.	5¼ hrs.	9½ hrs.	½mm.	Sound	Sound	Sound	Sound'	½mm.
B. ENGLISH ROTARY KILN CEMENT	2·8	nil	9·6	2·8	nil	2¼ hrs.	6½ hrs.	3½ hrs.	7 hrs.	4½ hrs	7½ hrs.	4 hrs.	7½ hrs.	0mm.	do.	do.	do.	do	0mm
C. ENGLISH CHAMBER KILN CEMENT	8·2	0 2	16·0	8·2	0·2	14 mins.	30 mins.	18 mins.	30 mins.	20 mins.	35 mins.	20 mins.	40 mins.	1mm	do.	do.	do	do	1mm
D. "CIMENT ELECTRIQUE"	1·7	1·0	4·4	1 7	1·0	2½ hrs.	7 hrs.	4⅜ hrs.	9 hrs.	7½ hrs.	11 hrs.	5 hrs.	9 hrs.	0mm	do	do.	do	do	0mm

tensile strain than the British Standard Specification method, both at 7 and 28 days.

The Argentine Government Specification calls for a wetter mixture still, the percentage of water being determined by means of the Tetmajer plunger, i.e., a cylindrical rod of 1 c.m. diameter, weighted to 300 grams; the neat cement paste is placed in a mould 8 c.m. diameter and 4 c.m. deep, and smoothed off with a trowel. When the weighted rod or plunger, placed on the top of the mass, sinks to within 6 mm. of the bottom the paste is considered to be of normal consistency. The briquette moulds are filled by placing an excess of paste in the moulds at one operation; this is pressed in with the fingers, and the sides of the moulds tapped with a trowel in order to eliminate air bubbles, and the excess of paste is removed with a straight-bladed knife and the surface of the briquettes smoothed off with the same instrument.

The greater percentage of water and method of filling the moulds called for by the Argentine Specification result in an even lower tensile strength than by the U.S.A. method. In the author's opinion the somewhat indefinite method of determining the amount of water used for gauging is a weak point of the British Standard Specification. The clause dealing with this point reads: "The cement shall be mixed with such a proportion of water that the mixture shall be plastic when filled into the moulds"; the same clause also regulates the gauging of the neat cement for determining setting time. This leaves the interpretation of the word "plastic" entirely to the operator, and it is conceivable that the ideas of different operators on this point may be widely different. For the sake of uniformity, it would be desirable to adopt some such exact method of determining the percentage of water as that of the Argentine Government Specification, though if a wetter mixture were adopted allowance would have to be made for the lower tensile strength developed due to the greater percentage of water used; the setting time would also be slower under the latter conditions.

The methods of preparing 3 : 1 sand-cement-mortar briquettes, prescribed by the three specifications in question, differ even more widely than that of

TABLE No. VI.

Tests for Tensile Strength

Of four different cements under British Standard Specification, U.S.A. Government Specification, and Argentine Government Specification conditions.

Average of six briquettes in each case.

Sample.	Neat Cement.									3 : 1 British Standard Sand Cement Mortar.								
	Specification.									Specification.								
	British Standard.			U.S.A.			Argentine.			British Standard.			U.S.A.			Argentine.		
	Water used for gauging.	Date of Testing in days.		Water used for gauging.	Date of Testing in Days.		Water used for gauging.	Date of Testing in Days.		Water used for gauging.	Date of Testing in Days.		Water used for gauging.	Date of Testing in Days.		Water used for gauging.	Date of Testing in Days.	
		7	28		7	28		7	28		7	28		7	28		7	28
	%	lbs.	lbs.	%	lbs.	lbs.	%	lbs.	lbs.	%	lbs.	lbs.	%	lbs.	lbs.	%	lbs.	lbs.
A. Belgian Natural Cement	22·65	506	611	30·0	358	472	31·33	329	498	8·33	286	333	11·5	264	279	9·37	340	410
B. English Rotary Kiln Cement	21·66	793	900	27·5	597	789	28·33	612	736	8·33	402	477	11·11	343	432	8·75	490	594
C. English Chamber Kiln Cement	22·5	565	678	28·66	451	580	30·0	406	536	8·33	317	419	11·11	226	360	9·5	390	483
D. Ciment Electrique	20·0	1033		26·87	626		30·0	506		8·33	489		10·85	355		9·37	435	

moulding the neat briquettes, with correspondingly different results, as shown in the table. The method giving the lowest result is that of the U.S.A. Specification, in which the largest percentage of water is used, and the method of filling the moulds results in the least compact briquette.

By the British Standard Specification regulations the percentage of water used is again somewhat indefinite, depending, as with the neat briquettes, chiefly upon the personal view of the operator; the clause reads—" shall be gauged with sufficient water to wet the whole amount throughout, without any excess of water being present."

With both the U.S.A. and the Argentine Specifications the percentage is governed by the amount of water required to produce a cement paste of "normal consistency," which latter is determined in both cases by slightly different applications of the Tetmajer plunger.

In the U.S.A. Specification a table is given showing the amount of water to be used for 3 : 1 sand-mortar briquettes according to the percentage required for producing neat cement paste of normal consistency, which normal consistency is also used for determination of setting time. This table ranges from 9 per cent. for the mortar when the neat cement requires 15 per cent., up to 11½ per cent. for the mortar when the neat cement figure is 30 per cent.

By the Argentine Specification the quantity of water to be added to 1 kilogram of 3 : 1 mortar is regulated by the formula 45 + 1/6 P, "P" representing the weight of water necessary to reduce 1 kilogram of neat cement to a paste of normal consistency.

In moulding the sand briquettes, the British Standard Specification provides that "the mixture shall be evenly distributed in the moulds," and that "after filling a mould a small heap of the mixture shall be placed upon that in the mould and patted down with the standard spatula . . . until the mixture is level with the top of the mould. This last operation shall be repeated a second time and the mixture patted down until water appears on the surface; the flat only of the standard spatula is to be used, and no other instrument or apparatus is to be employed for this operation. The mould after being filled may be shaken to the extent necessary for expelling the air. No ramming or hammering in any form will be permitted during the preparation of the briquettes, which shall then be finished off in the moulds by smoothing the surface with the blade of a trowel."

The U.S.A. Specification, on the other hand, stipulates that "the standard mortar shall be placed in the moulds, pressed in firmly with the thumbs and smoothed off with a trowel without ramming. Additional mortar shall be heaped above the mould and smoothed off with a trowel; the trowel shall be drawn over the mould in such a manner as to exert a moderate pressure on the material. The mould shall then be turned over and the operation of heaping and smoothing off repeated."

In this operation there is no beating in of the mortar with the spatula or other instrument, only thumb pressure and smoothing off with the trowel, the result being that, in combination with the wetter mixture, the briquette is considerably less dense than by the British Standard Specification method, and consequently gives a lower result.

With the Argentine Specification the mortar is mixed by machine, and beaten into the moulds by means of a Böhme hammer apparatus, described as follows: "The moulds shall be filled with an excess of mortar. This shall be compressed into the moulds by means of an apparatus, in which a hammer of 2 kilograms in weight shall fall 120 consecutive times from 40 centimetres height. The excess of mortar shall be extracted by means of the blade of a knife which also serves to smooth the surface of the briquette."

Referring to the comparative results of tests, in each case the U.S.A. method gives lower results than the British Standard Specification, while the Böhme hammer method of mechanically beating the mixture into the moulds gives considerably higher results than either of the other two methods.

Although, as before stated, modern Portland cement develops a strength far in excess of the minimum requirements of the British Standard Specification, the results given in Table V show that a cement which only just comes within the minimum requirements of the British Standard Specification would fail to pass both the U.S.A. and Argentine Specifications, i.e., under the sand test with the former and the neat test with the latter. Under the British Standard Specification conditions 200 lb. per sq. in. at 7 days would be equivalent to only about 170 lb. under the U.S.A. Specification, while the 250 lb. minimum at 28 days would obviously not reach the 300 lb. required under the U.S.A. conditions; on the other hand, the British Standard Specification minimum of 450 lb. at 7 days for neat cement would be equivalent to only about 316 lb. under the Argentine Specification conditions.

In these two respects, therefore, the British Standard Specification is considerably weaker than the U.S.A. and Argentine Government specifications, which, in the author's opinion, is a further argument in favour of its being made very much stiffer, so that cement reaching the minimum conditions of the British Standard Specification shall be equal to the minimum conditions of the specifications of other countries.

A rather striking feature of Table V is the very marked difference that the wetter mixture and different method of preparing neat briquettes has upon the *ciment electrique*, reducing its strength, in the case of 30 per cent. of water, by fully 50 per cent. as compared with the British Standard Specification consistency. The same remark applies, though not quite to such a marked extent, to the sand tests by the U.S.A. method; it is scarcely fair, perhaps, to judge this cement by ordinary Portland cement standards, since it is a cement of a totally different type, both chemically and physically, its chief characteristic being its rapid hardening properties, attaining in 48 hours substantially the same strength as most Portland cements would develop in 28 days.

"*Ciment electrique*," "*ciment fondu*," or "Alca cement," to give it its various names, is essentially a high alumina cement and almost black in colour; its chief chemical constituents, as determined from a sample submitted two years ago, are as in Table VII, and for comparative purposes the chief constituents of a typical Portland cement are placed alongside.

TABLE VII.

	Ciment Fondu.	Portland Cement.
Insoluble residue ..	1·74	·26
Silica	10·10	23·31
Alumina	39·46	6·38
Oxide of iron ...	5·35	3·10
Lime	42·55	63·27
Magnesia	0·57	1·16
Sulphuric anhydride	·10	1·14

In the manufacture of Portland cement the intimately blended raw materials are heated to clinkering temperature, i.e., incipient vitrification only, and would be rendered altogether useless if carried to complete fusion. With "*ciment fondu*," on the other hand, the mixed raw materials are fused in a water-jacketed kiln, or in an electric furnace, the subsequent operation of grinding the calcined mass to a very fine powder being the same as with Portland cement.

The chief characteristic of "*ciment fondu*" is its rapid hardening properties; tests made by the author some six months ago, with a sample submitted by users in this country, gave the results shown in Table VIII.

It is also claimed for it that it is immune from attack by sea water and water containing sulphur compounds, which have sometimes been responsible for unsatisfactory work where Portland cement has been employed, and experiments by responsible French engineers are put forward in support of this claim. Its rapid hardening qualities should render it very valuable for constructive work, such as street repairs, etc., where time is the chief consideration, but until means are found of producing it more cheaply (the cost of production at present being somewhere about double that of Portland cement) it does not appear likely to become a serious rival of the latter for ordinary constructive work.

TABLE VIII.

	Tensile Tests.		Crushing Tests.	
	Lbs. per sq, in		Lbs per sq. in.	
	Neat	3-1 Sand	Neat	3-1 Sand.
1 day ...	602	253	6,777	2,814
2 days ...	833	373	7,741	4,414
7 days ..	933	512	9,089	5,810
28 days ..	1,030	563	12,196	6,596

In conclusion, the author would like to summarise briefly the points set forth in the paper in support of his contention that the current British Standard Specification requirements need revision and strengthening. Broadly speaking, they are to the effect that modern British Portland cement is so far superior to the minimum requirements of the British Standard Specification that to keep it at its present low level is to encourage the manufacture of an inferior material. British manufacturers have a well-earned reputation for quality, and anything which would tend to lower that quality cannot be too strongly deprecated. British manufacturers have greatly improved their product during the past few years, and at the present time there is every justification for the statement that, on the average, British Portland cement is equal to, if not better than, that of any other country.

The author is, therefore, of opinion that considerably stiffening the conditions of the British Standard Specification would be to the best interests of British manufacturers, and would tend to maintain the high reputation which British manufactures now enjoy throughout the world.

DISCUSSION.

The Chairman (Mr. H. J. Deane) said the great justification for making the British Standard Specification had been that in years gone by there were so many different ideas as to what cement should be that it became necessary to try to absorb them under one cover. A question arose as to what was aimed at in getting a better specification, bearing in mind that there were so many different kinds of work for which cement was required. It might be considered whether it was desirable to have more than one specification—one for the finest work and another for the roughest work. A cheap cement would be a desirable thing for rough work.

Mr. Wentworth Sheilds said he agreed that the British Standard Specification required stiffening. He agreed that the fineness of grinding should be reduced, say, from 14 to 10 per cent., and the tensile tests should be stiffened; the neat test at seven days might be put up to something like 600 lb. per sq. in., and the sand test to 300 lb. per sq. in. The Americans had abolished the neat test altogether, and it was sometimes suggested that we should do the same. He thought it would be a pity to do this, but when one got a high tensile test at seven days it was sometimes difficult or impossible to get the increase required by the British Standard Specification at 21 days. He thought the figures given for *ciment fondu* did not bear out all that was claimed for it. The lecturer's tests showed that the sand briquette developed a strength of 373 lb. per sq. in. at two days; but they also showed that that figure could be beaten by good British cement at seven days.

Mr. Charles Marsh said he agreed with what the lecturer had said with regard to the strengthening of the British Standard Specification, and as a matter of fact in the contracts with which he had had to deal lately he had inserted an addition to the clause governing Portland cement providing for an increase in regard to the fineness of grinding. The electric cement had been originally experimented with in 1908, and had been practically applied by the French Government engineers in 1914. He understood that early this year this cement had been quoted at about £10 a ton. It seemed to him it was a cement which would not be used in the ordinary course at all, but would be suitable where special precautions were necessary; as when one wanted to strike centering very quickly.

Mr. T. J. Guéritte said if one fixed 600 lb. for the neat cement test there would be a certain amount of danger in fixing 300 lb. for the sand test; because, although it was correct to say that the sand briquettes gave an average of 50 per cent., there was a very wide margin of variation. In Table 3, taking the aggregate of all the twenty-five tests, one obtained exactly 50 per cent. for the value of the seven days' sand compared with the seven days' neat; but if the percentage was worked out in every case it was found it varied from 35 per cent. in one case to 64 per cent. in another. A point in which he was in hearty agreement with the lecturer was as to the desirability of doing away with the looseness of the wording of the British Specification in some respects.

Mr. H. K. Bamber said British cement to-day was about 100 per cent. better than British Standard requirements. But consumers, alike in this country and abroad, did not fully appreciate the improvement that had taken place; and the result was that abroad foreign made cements found a market although they only just passed the British Standard Specification. He would prefer to abolish the neat cement tensile test altogether—as had been done by other nations manufacturing Portland cement—but if the test were retained 600 lb. was a reasonable figure. Even at that standard he believed the consumer would be obtaining cement considerably above the requirement. As regarded the sand test, 300 lb. was a reasonable figure. It was one the cement industry should entirely agree to, and the Sectional Committee on Cement of the British Standards Association was willing that the Specification should be increased to that extent. The British Specification left the question of the water to be used for gauging rather open to the opinion of the individual, and he would prefer that some method should be specified for determining the percentage of water for each class of cement, as was done in other countries. But if an alteration were made which resulted in a larger quantity of water being used they must remember that there would be a reduction in the strength of the briquette produced, and must adjust the tests accordingly. As to *ciment fondu*, he would give some figures showing the results of comparative tests of this cement with two brands of Portland cement. Tested with three parts of sand, in accordance with the British standard method, *ciment fondu* at 48 hours had a tensile strength of 427 lb. Two brands of Portland cement under exactly the same conditions had 380 lb. and 360 lb. At three days *ciment fondu* had a strength of 450 lb., and the two brands of Portland cement 394 lb. and 445 lb. At seven days *ciment fondu* had a strength of 473 lb., one of the Portland cements 524 lb., and the other 450 lb. At 28 days the figures were: *ciment fondu* 510 lb. and the two Portland cements 550 lb. and 617 lb. He could not conceive that *ciment fondu* was going to be a very strong competitor with the best brands of Portland cement, except on those occasions when it would be worth while to pay any price to get a cement which would harden in forty-eight hours for some special purpose.

Mr. H. G. Lloyd said a practical objection to very finely ground cement on exposed works was that it was more liable to be blown away than a coarser quality. Comparing some figures for the early part of last year with those of this year he had found that for corresponding grindings this year's cement tended to give higher results. Undoubtedly the amount of water put into the cement made a considerable difference to the tests: one could get a regular percentage result of tensile strength depending entirely upon the percentage of water put in. When testing he preferred the Boulogne ball method to any method where a plunger was used. One could feel exactly what the cement was like, and could tell to within a half of 1 per cent. when a consistency was obtained which held the cement very firmly together and reduced it to a rubbery state.

Mr. H. Brooks said laboratory practice could never be approximated to commercial practice, but it was convenient to arrive at some standard method by which relative values of cement could be tested. A cement possessing certain qualities was capable of giving certain results in ordinary practice, and the object of the British Specifications Committee was to lay down a standard of laboratory practice which would show whether a particular cement offered in commerce would in practice give certain results. He did not agree with the lecturer that the coarse particles of cement were practically inert, although in the early stages of testing a coarse cement did not by any means develop its full strength. If a finer grinding in the Standard Specification were asked for they would get a more rapid development of the strength which that cement possessed, but they must look for a smaller increase of strength at later dates. He agreed that neat cement tests should be abolished. He favoured the German practice of abolishing tensile tests, but when that question had been last discussed at the British Standard Specifications Committee it was pointed out that the tensile method had certain advantages. The laboratories of this country, and generally of the world, were equipped with tensile testing machines; these were cheaper to construct than compression test machines, and if the compression test only were adopted testers and practitioners would be involved in an enormous expense for new and more costly machines. At the same time compression was the real test to which cement should be put. Speaking as a manufacturer, he would say that consumers could have any fineness of cement that they were prepared to pay for.

Mr. H. T. Desch said he had read that afternoon a specification, by an eminent engineer, which stated that the cement was not to be used until it was six weeks old. It was to be taken out of the sacks, put into a cement shed, and turned over as many times as the engineer thought fit. It seemed to him that some one had been to sleep for a good many years; carrying out such a specification would injure the cement.

Mr. G. A. Gardner said one question bearing upon the question of the uniformity of conditions of test was the exact shape of the briquette. Professor Coker had found that the tensile strength was decidedly uneven across the section in the case of the British shape briquette, and even more so in the case of the French briquette. Some remarks had been made with regard to doing away with the tensile test; but whilst we usually turned to the compressive values for reinforced concrete design, exact data as to tensile value were not to be neglected in view of the important part played by the tensile resistance of concrete in reinforced concrete shells subject to flexure, as in the case of tall chimney stacks, especially where the engineer thought it desirable to limit the resistance of the section to one compatible with a homogeneous function of steel and concrete.

CONCRETE TANKS FOR HOT LIQUIDS AND ACIDS.

Concrete tanks built for the production of alcohol from the waste sulphite liquor, which is a by-product in the manufacture of cellulose, are described in a recent issue of *Beton u. Eisen*. The tanks are cylindrical, with flat bottoms, and are covered with conical roofs. They are used to contain liquor at a temperature of 100° C., whilst the external temperature may be well below freezing point. The raw liquor passes from these tanks into neutralising towers, in which the sulphurous acid is neutralised by means of chalk. The towers are 36 ft. high, and have conical bottoms. The liquor in them attains a temperature of 90° C., but the outside of the towers may be as low as 10° C.

The neutralised liquor is allowed to stand in rectangular settling tanks in order to separate the calcium sulphite, the temperature in these tanks being about 60° C. After passing through concrete cooling towers, in which the temperature is reduced to 30° C., the liquor flows into open fermentation tanks, in which it is maintained at 30° C. for 4 days, during which it develops large volumes of carbon dioxide. The alcoholic liquor thus produced is then passed into a continuous still.

Hitherto, the customary material for tanks to contain hot liquids was iron, and for those to contain acid liquids copper or lead was employed.. The chief difficulty lies in making proper allowance for the effect of expansion and contraction of the cement, but by making the walls of sufficient resistance to vertical and horizontal forces perfectly sound tanks were produced.

Concrete Road at Cambridge.

The tender of Messrs. Arthur Negus and Sons has been accepted by Jesus College, Cambridge, for the construction of a new road from Cherryhinton Road to the old Newmarket railway. The new road, which is to be of unsurfaced reinforced concrete, will be 50 ft. in width, with a 24-ft. carriageway, a grass strip 5 ft. in width, and a footway 8 ft. in width on each side. The sewers and surface water drains, and gas, electric light and water supplies will be laid under the grass strips, so as to obviate the necessity for disturbing the surface of the road.

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

***Readers are cordially invited to send in questions relating to concrete, which will, as far as possible, be answered direct and subsequently published where they are of general interest. We cannot give complete designs; these are the province of the consulting engineer. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.*—Ed.**

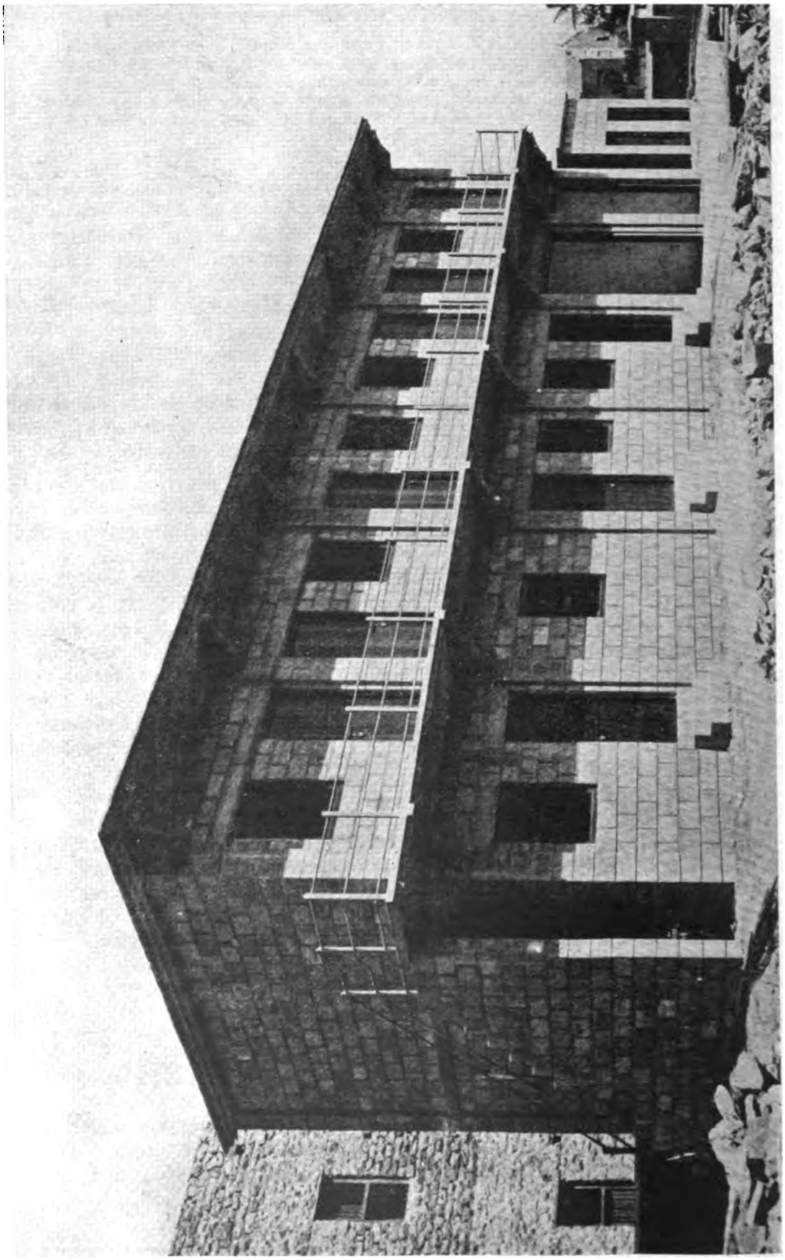
RESERVOIR DESIGN.

Question.—In the case of a covered-in reservoir I take it that it is correct to design the sides as simple beams supported at the top and bottom, or with vertical beams and the slab reinforced horizontally and carried by these beams. The reaction at the top then being $\frac{1}{6}\,ph^2$, and at the bottom $\frac{1}{3}\,ph^2$, and the maximum moment $\cdot064\,ph^3$ occurring at a distance from the top of $\cdot58\,h$. I presume the horizontal roof beams taking the top reaction will have to be designed for combined bending and tension, but is it permissible to assume that the bottom reaction is taken by the ground slab of the reservoir? This slab might only be, say, 6 in. thick, with the addition of a 6-in. fillet. Also, will the vertical wall beams require to be designed for combined bending and compression, i.e., to take the reaction from the roof beams? If it is not permissible to assume the ground slab as taking the reaction, and a base slab to the wall is provided, is this base slab to be considered as taking the whole of the overturning moment, and support at the top neglected altogether? If for some reason the size of this base slab is restricted, and is not sufficient to resist overturning or excessive pressure on the foundation results, can the base slab be assumed to take an overturning moment sufficient to meet with the ground pressure requirements, and the balance of the overturning moment taken by the top beams? I have an idea that I have seen something of this nature done, but it seems to me that once any support at the top is reckoned on overturning cannot occur, it becoming simply a question of a beam, the top and bottom reaction being horizontal and the load on the foundation uniform.—" N.V.H."

Answer.—The position outlined in the first sentences is substantially correct. The sides being continuous with the top and bottom, the moment and, to a much lesser extent, the reactions, will be altered from the figures given. It is correct to state that the roof beams should be designed for combined bending and tension, and the bottom reaction may be taken by the ground slab as suggested provided this is suitably reinforced. The vertical walls will be subject to combined bending and compression as stated, the latter, however, frequently being almost negligible, depending, of course, on the general design and proportions of the reservoir. As the walls may be considered as tied in by roof and base, the questions in the latter part of our correspondent's letter do not arise.

SERVANTS' QUARTERS AT KARACHI.

The illustration on page 558 shows a small block of servants' quarters which has just been constructed for the Karachi branch of the Alliance Bank of Simla. The floors are of reinforced concrete. The walls are built of " Winget " concrete blocks.



SERVANTS' QUARTERS AT KARACHI, FOR THE ALLIANCE BANK OF SIMLA. (*See p.* 557.)

[*Mr. E. B. Hoare, F.R.I.B.A., Architect.*

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Portland Cement	10s. per ton extra		

BOARDING FOR SHUTTERING—		Sawn.		Wrot.	
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—	
3 in. by 4 in.	from £24 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £25 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	14	9
7/16 in. to ½ in. Rounds	,,	14	6
⅜ in. Rounds.	,,	15	6
¼ in. Rounds.	,,	17	6

MATERIAL AND LABOUR INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED WITH CLEAN WATER IN PROPORTION.

1 : 2 : 4—				s.	d.
Do.	do.	in foundation	per yard cube	45	0
Do.	do.	in columns	,, ,,	50	0
Do.	do.	in beams.	,, ,,	50	0
Do.	do.	in floor slabs 4 in. thick	per yard super	5	3
Do.	do.	in floor slabs 6 in. thick	,, ,,	7	9
Do.	do.	in floor slabs 9 in. thick	,, ,,	11	3
Do.	do.	in walls 6 in. thick	,, ,,	8	0

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—		s.	d.
From ¼ in. to ⅜ in.	per cwt.	27	0
,, 7/16 in. to ½ in.	,,	25	0
,, ⅝ in. to 2⅞ in.	,,	24	0

SHUTTERING—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	54	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	45	0
Do. do. in small quantities	per ft. super	0	9
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	10
Do. do. as last in narrow widths	,, ,,	1	0
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	0
Do. do. as last in narrow widths	,, ,,	1	2
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

WAGES.—The rates of wages on which the above prices are based are those current in the London area, namely:—Carpenters and joiners, 1/7½ per hour; Labourers on building works, 1/2¾; Operators on concrete mixers and hoists, 1/3¾.

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

MEMORANDA.

PROSPECTIVE NEW CONCRETE WORK.

BARGOED.—*Houses.*—The Duffryn Steam Coal Co., Ltd., intend erecting 1,250 houses on the Heolddu Uchaf estate and further houses on the Pencaedrain Farm site.

BARMOUTH.—*Railway Station.*—Substantial alterations are to be carried out by the G.W.R. Co., at a cost of about £25,000, including the lengthening of the up platform from 480 ft. to 700 ft., and the provision of a new up bay platform.

BARNSLEY.—*Sewage.*—The Unemployment Grants Committee has approved of the Barnsley scheme for the centralisation of the sewage works of the borough, involving an expenditure of £21,650.

BARRY.—*Marine Lake.*—The U.D.C. awaits the decision of the Unemployment Grants Committee respecting a marine lake which is estimated to cost £60,000.

BENTLEY.—*Sewage.*—The U.D.C. has decided to apply to the M.H. for permission to borrow £14,350 for sewage works extensions.

BIDEFORD.—*Bridge.*—At a recent meeting of the Devon County Bridges and Main Roads Committee, it was resolved to recommend the C.C. to construct a new bridge of concrete across the river Torridge at Bideford, at an estimated cost of £3,250.

BOURNEMOUTH.—*Shelter.*—The Corporation has decided to erect a shelter in the square, at a cost of £3,000.

BOURNEMOUTH.—*Road.*—The House of Lords has passed the Bill containing the proposals of the Bournemouth-Swanage Motor Road and Ferry Co. The company, whose proposed capital is to be £60,000, with authority to borrow a further £30,000, has been seeking power to construct a motor road, approximately two miles seven furlongs in length from South Haven Point to the public road leading from Studland to Swanage.

BRIDLINGTON.—*Road.*—The T.C. has resolved to apply to the M.H. for sanction to borrow £1,800, the estimated cost of the acquisition of the land required for the proposed new road from St. John's Avenue to Midway Avenue.

BRUSSELS.—*Dock.*—It is announced that £625,000 is to be spent in adding a dock for large ships to the port of Brussels.

CALCUTTA.—*Docks, etc.*—It is stated that the Commissioners for the Port of Calcutta have been authorised to raise a loan of £1,000,000 in London to meet expenditure on port works.

CAPE TOWN.—*Breakwater.*—In the estimates laid before the Union Parliament by the Harbours and Railway Board, provision is made for the extension of Cape Town Breakwater by 1,500 ft.

CHEPSTOW.—*Sewage.*—An inquiry has been held into the application of the R.D.C. for sanction to borrow £4,000 for sewage disposal.

DEWSBURY.—*Road.*—The Corporation has agreed to a scheme for the construction of a new road from Leeds Road to Wakefield Road, at an estimated cost of £30,000.

DONCASTER.—*Houses.*—Developments in the coal industry at Doncaster are about to take place. At Costhorpe, near Worksop, shaft sinking operations have been commenced. About 1,000 houses will be needed shortly, and it is said that £20,000,000 is waiting to be invested within ten miles of Worksop.

EDGWARE.—*Station.*—It is announced that the Underground Railway Co. is to construct a station at Edgware at a cost of £16,690.

ELY.—*Houses.*—The Cardiff Corporation is considering a scheme for the erection of 2,000 houses at Ely.

EXETER.—*Road.*—The Corporation has appointed a committee to consider the construction of a loop road in the outskirts of the city.

GERALDTON.—*Harbour.*—The Western Australian Government has agreed to a harbour development scheme at Geraldton, involving an expenditure of £400,000.

GREAT HARWOOD.—*Sewage.*—The Great Harwood and Clayton-le-Moors sewage works are to be modernised at an estimated cost of £500,000.

HARWORTH.—*Sewage.*—The Blyth and Cuckney (Notts.) R.D.C. has decided to prepare a sewage disposal scheme for the village of Harworth.

LEEDS.—*Roads.*—A scheme has been submitted to the C.C. for work for the unemployed during the next winter. The estimated expenditure, £398,000, includes new roads, extensions to sewers, extension to reservoir at Moortown, etc.

LONDON (CATFORD).—*Houses.*—The L.C.C. has authorised the Housing Committee to invite tenders for the development of the Bromley Road section of the Grove Park housing site. The section comprises about 183 acres, on which it is estimated that some 2,000 houses can be built.

LONDON, W.—*Building.*—It is stated that the Automobile Association and Motor Union are to build new headquarters in Leicester Square at a cost of £115,000.

LYTHAM.—*Houses.*—The T.C. is to erect seventy-two houses, at a cost of £36,000.

MERTHYR TYDFIL.—*Houses.*—The T.C. has received sanction to the erection of fifty houses, which will be built by direct labour.

NARBETH.—*Bridge.*—A bridge, 20 ft. wide, is to be erected at Felinfach by the Narbeth R.D.C., at a cost of £700.

NORTHENDEN. — *Sewage.* — The Bucklow R.D.C. has applied to the M.H. for sanction to borrow £13,000 for the reconstruction of the sewage disposal works.

SEAHAM.—*Dock.*—A Bill before Parliament seeks powers to increase dock accommodation at Seaham Harbour, at an estimated cost of £180,000.

SHEFFIELD.—*Houses.*—The Corporation Estates Committee proposes to build 2,000 working-class houses.

SOMERSET.—*Road.*—The C.C. has decided to improve the Bristol Road between Puxton and Worle, by constructing a loop road. The estimated cost is £9,000.

SOUTH SHIELDS.—*Houses.*—The T.C. has agreed to build a further 600 houses on the Cleandon Park estate.

STOCKTON.—*Roads.*—The Borough Surveyor has submitted alternative schemes for a new road from Tilcry to Portrack, one estimated to cost £10,000 and the other £6,500.

SUDBURY.—*Sewage.*—The T.C. proposes to spend £1,000 on improvements at the sewage disposal works.

SUTTON.—*Sewage.*—The U.D.C. has applied to the M.H. for sanction to borrow £1,900 for sewage disposal works.

TORQUAY.—*Sea Wall.*—The Corporation has decided to construct a sea wall between Abbey Crescent and Corbyn Head at a cost of £32,000.

WESTHOUGHTON. — *Sewage.* — The U.D.C. has applied to the M.H. for sanction to borrow £3,100 for sewage disposal works at Roger's Farm and Marsh Brook Farm.

WILLIAMSTOWN.—*Dry Dock.*—The Melbourne Harbour Trust has purchased Williamstown Dockyards from the Commonwealth Government for £150,000, and proposes to construct a dry dock capable of accommodating the largest liners.

YORK.—*Roads.*—The C.C. has agreed to replace the cobble paving in the back roads with reinforced concrete, at an estimated cost of £18,000.

TENDERS ACCEPTED.

ABERDEEN.—*Bridge, etc.*—The T.C. has accepted the tenders of Messrs. Clark and Chapman, Aberdeen, at £4,699 8*s.* 4*d.* for the erection of a reinforced bridge, and £1,899 16*s.* 5*d.* for the construction of reinforced concrete flood-water culverts.

AUCKLAND.—*Sewage.*—The R.D.C. has accepted the tender of Mr. H. C. Eisel, Coundon, at £126 18*s.*, for sewage disposal works at Byers Green.

BELFAST.—*Houses.*—The T.C. has accepted the tender of Messrs. Weir & Wilson to erect seventy concrete houses on the Ulsterville site at £438 5*s.* per house. Other tenders received included: J. and R. Thompson, Ltd., £462 per house, and John Rainey, (for 21 houses) £507 per house.

BLYTH.—*Pier.*—The Blyth Harbour Commissioners have accepted the tender of Messrs. Brims and Co., Ltd., for the reconstruction of the West Pier. The new pier is to be of reinforced concrete, 1,300 ft. long.

BRANSFORD. — *Bridge.* — The Worcestershire C.C. has accepted the tender of Messrs. T. Vale and Sons, Ltd., Stourport, at £9,255 9*s.* 6*d.*, for the construction of a new ferro-concrete bridge over the River Teme.

CLYDACH.—*Reservoir.*—The Crickhowell R.D.C. has accepted the tender of Mr. G. A. Brown, Ebbw Vale, at £313 13*s.* 4*d.*, for the construction of a reinforced concrete reservoir at Clydach.

DEWSBURY.—*Sewage.*—The T.C. has accepted the tender of Messrs. Wellerman Bros., Ltd., at £6,145 5*s.* 6*d.*, for the construction of sewage disposal works.

DOVER. — *Steps.* — The Corporation has accepted the tender of Messrs. G. Lewis & Son, at £175, for concrete steps at the pavilion of the Crabble Athletic Ground.

LONDON.—*Foundation.*—The L.C.C. has accepted the tender of Mr. F. R. Hipperson, Norwich, at £13,815 2*s.* 8*d.*, for the construction of foundations for machinery at Abbey Mills pumping station.

LONDON.—*Raft Foundation.*—The L.C.C. has accepted the tender of Messrs. Holloway Bros., at £58,126, for the construction of the raft foundation, retaining walls, etc., for the northern extension of the County Hall.

OSSETT, YORKS.—*Water Tower.*—The T.C. has accepted the tender of Messrs. J. Wimpenny & Co., Huddersfield, at £7,300, for the erection of a reinforced concrete water tower with a capacity of 200,000 gallons.

SHEFFIELD.—*River Wall.*—The T.C. has accepted the tender of Messrs. A. Waddington & Son, at £1,439 5*s.*, for the rebuilding of the river retaining wall at Smithfield Market.

WALTON-LE-DALE.—*Reservoir.*—Subject to the consent of the M.H., the U.D.C. has provisionally accepted the tender of the Leyland Construction Company for a ferro-concrete reservoir at £3,997.

WHITEHAVEN. — *Reservoir.* — The R.D.C. has accepted the tender (subject to amendments) of Messrs. Grays Ferro-Concrete Co., Glasgow, at £4,033 11*s.* 5*d.*, for the extension of Cold Fell water supply, parish of St. Johns.

RECENT PATENT APPLICATIONS.

198,129.—J. S. Haslam: Concrete mixers.

198,161.—B. Heffer: Concrete pipe-making machines.

198,244.—K. Wettstein: Manufacture of moulded reinforced concrete slabs.

198,624.—Messrs. H. Lobban and C. A. Geneve: Reinforced concrete floors.

198,783.—L. Alwyn: Manufacture of drawbridges, cranes, and dock gates.

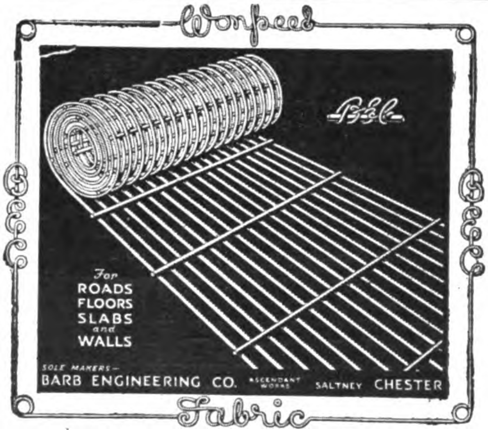
198,896.—J. G. Zwicker: Apparatus for moulding pipe sections.

NEW COMPANIES REGISTERED.

Centrifugal Concrete Products, Ltd. (191,067). Registered July 3, Lower Belgrave Street, S.W.1. Manufacturers and vendors of reinforced and other concrete articles, etc. Qualification of Directors: 1 share. Nominal capital, £2,000 in 2,000 shares of £1 each. Directors to be appointed by subscribers. Remuneration of directors to be voted by company.

Girling's Ferro-Concrete Co., Ltd. (190,823). Registered June 21, New Howley Park, Morley. Concrete specialists, builders and contractors, etc. Nominal capital, £3,000 in 3,000 shares of £1 each. Directors, Messrs. M. and R. P. Girling, The Mount, Woodkirk, York. Qualification of directors, £100; Remuneration to be voted.

Manufactured Concrete Works, Ltd. (190,417). Registered June 4, 39, Victoria Street, S.W.1. Manufacturers of concrete or reinforced concrete articles. Nominal capital £5,000, in 5,000 £1 shares. Directors to be appointed by subscribers. Remuneration of directors to be voted by company. Subscribers: P. Westacott, 39, Victoria Street, S.W.1; R. P. Campbell, 77, St. Stephens Avenue, Shepherd's Bush, W.



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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

SEPTEMBER 1923 VOL. XVIII. No. 9

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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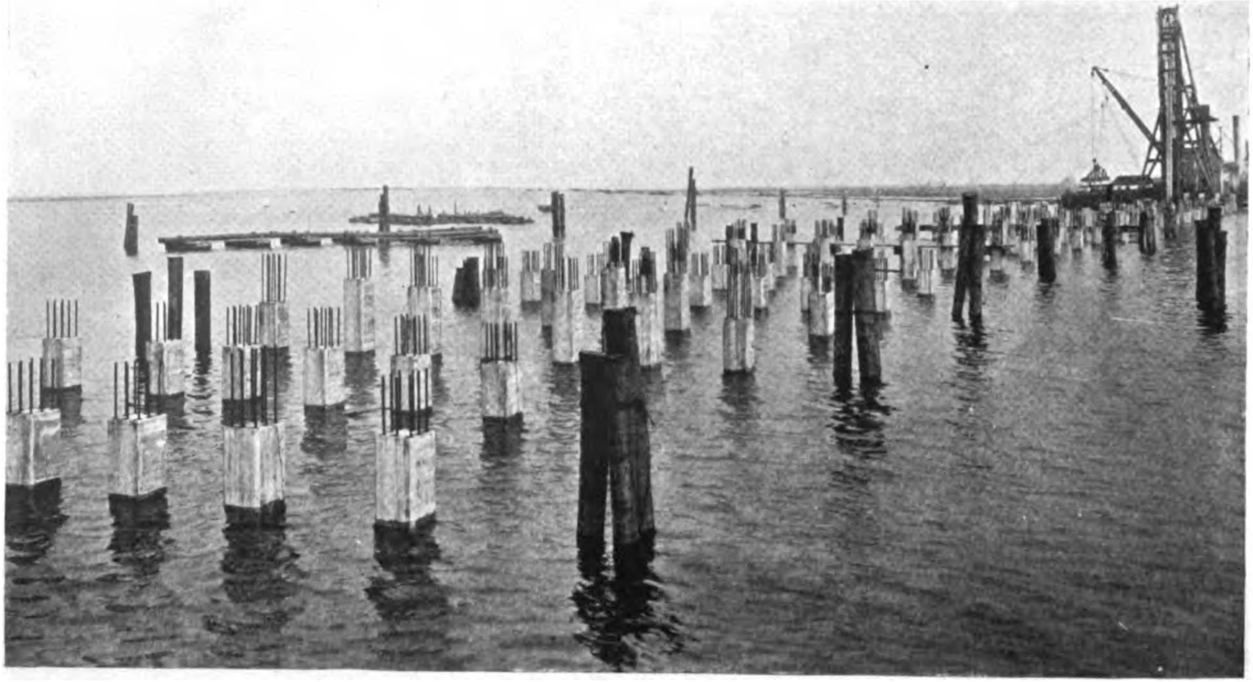

WOODHAVEN BOULEVARD EXTENSION, JAMAICA BAY, NEW YORK. (*See. p.* 595.)

General View after some of the 24-in. Piles had been Driven.

[This Extension will be about seven miles in length and 100 ft. wide when completed.]

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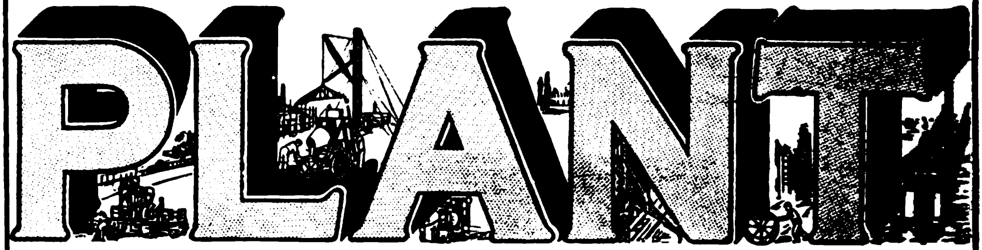

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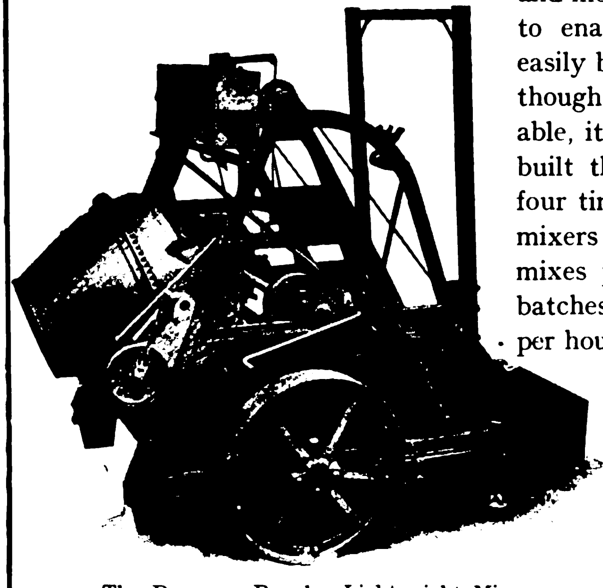

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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume XVIII. No. 9. LONDON, SEPTEMBER, 1923.

EDITORIAL NOTES.

NEW APPLICATIONS OF CONCRETE.

In our June issue we stated: "The variety of attractive finishes which can be imparted to the concrete itself, at little extra cost, is not generally known, and, of course, cannot be realised until there are in this country some examples of the processes from which the results can be judged by personal inspection; what is undoubtedly the cheapest method of imparting a decorative value to a modern commercial building is in danger of being neglected because pioneers are not forthcoming to adopt it." We are now able to illustrate (pp. 576–8) what is, we believe, the first concrete building to be erected in this country in which no other material has been added to obtain a pleasing texture. In certain types of building, such as the huge concrete structures now being erected at Wembley, effect may be obtained by design alone—in fact, it is probable that any colouring on such buildings would detract from, rather than add to, the monumentality and immensity on which they rely for their appeal to the eye. On certain other classes of buildings, however, any architectural effect which adds to the air of brightness or gaiety with which the structures are associated is to be welcomed. The sports pavilion illustrated, of which Messrs. Henry Tanner, FF.R.I.B.A., are the architects, shows, in a small way, what can be done in this direction by the use of concrete, and is an admirable reply to those who have contended that concrete is not suitable for architectural expression but is a material for constructional purposes alone, to be hidden out of sight whenever possible.

We are glad to see that the possibilities of concrete from an architectural point of view are now being more generally realised by the architectural profession. The concrete buildings at Wembley and the sports pavilion at Snaresbrook are excellent examples of concrete buildings of widely different size and for very different purposes; the first housing exhibits depicting the resources of the Empire, the latter providing accommodation for a sports club, and both designed by eminent architects. The use of concrete for another type of building was referred to recently by Alderman W. C. Fenton, F.R.I.B.A. (Lord Mayor of Sheffield), who, speaking at the Annual Congress of the Royal Sanitary Institute at Hull, said: "One method of construction in which I have taken great interest is reinforced concrete, which I have found most useful for ordinary buildings as well as for engineering structures. I have had the opportunity of using it for quite large buildings . . . and I am shortly to put in hand a large block of offices and warehouses constructed entirely of reinforced concrete. There is no doubt that fine or refined mouldings can be easily worked in this material, and bold and striking mouldings can be well accomplished in this method of construction. . . . I feel certain that this form of construction has got possibilities before it, and that its use will become more general." He also referred to a church and vicarage in Yorkshire, which had been carried out entirely in reinforced concrete; these buildings, he said, had a most effective appearance, and no material could have been more suitable for the purpose.

It is gratifying to see this greater appreciation of the possibilities of the adaptation of concrete to artistic expression, although we have a long way to go before it is used as extensively here as in the United States. In our next issue we hope to illustrate and give some particulars of a large new church in Washington in which a leading firm of American architects have used concrete entirely for the interior decoration.

COMMERCIAL ARCHITECTURE.

THOSE critics who disagree with the modern system of architectural education, on the score that in the curricula of the schools the practical side of building construction is often made subservient to the study of the orders and draughtsmanship, will welcome the conditions of the "Alfred Bossom Travelling Studentship," recently issued by the Royal Institute of British Architects. The studentship, of the value of £250, will be awarded annually, the successful student being required to spend six months in the study of commercial architecture in the United States of America, and on his return to present a detailed report of his studies. The funds to meet the cost of the scheme have been generously provided by Mr. Alfred Bossom, a prominent New York architect, for five years, and it is to be hoped that at the end of that period it will be found possible to put the scheme on a permanent basis. It is certain that many of the new and improved methods of building construction now in common use in the United States could be profitably adopted in this country, and a signal service will be rendered if the reports of the successful students are given wide publicity so that they will be available for all concerned with building.

LARGE GLASS AREAS AND INSIDE TEMPERATURE.

SOME interesting experiments have recently been carried out at the new engineering works in Derby of the International Combustion Engineering Co., Ltd., the manufacturing company for the Underfeed Stoker Co., Ltd., of London. This factory probably contains the largest glass area, in the roof and sides, of any factory building in the world, and the experiments in question go to prove that this extensive glass area does not in any way cause undue heat inside the building, the temperature being actually less than in the shade outside.

The site of the factory is just outside Derby and covers 53 acres, the buildings comprising a block of drawing and general offices, a machine shop (226 ft. by 150 ft.), a foundry (262 ft. by 150 ft.), a pattern shop and pattern store and boiler house. In the machine shop, as an example, there is 33,561 sq. ft. of concrete floor area, whilst the total lighting glass in the structure is 34,117 sq. ft., the design including three cleaning gangways arranged in each span under the roof so that the glass can be cleaned easily.

On one day in July last during the investigations the temperature was 98° F.

in the sun, and 88° F. in the shade. During this time the average temperature over the whole of the machine shop was only 84·5° F., the maximum being 88° F. and the minimum 83° F., whilst in the foundries the average was 85·8° F., the maximum being 86° F. and the minimum 85° F. It will be seen, therefore, that the temperatures inside the factory during working hours were actually less than those outside in the shade, although no attempt, such as, for example, white-washing the glass or covering it with blinds, was made to prevent the sun's rays attaining their maximum effect, the whole factory being exactly as it is intended to work all the year round. Further, no mechanical means of circulation of the air inside the factories was resorted to, the results being obtained by the natural circulation of the air only, and the efficient method of the lay-out of the shops ; also the glass is of ordinary quality.

The advantages of the maximum amount of light in the way of increased efficiency and the health and well-being of the workers are obvious, and from this point of view, as well as on the ground of economy, the ideal method of factory construction would appear to be the provision of a maximum glass area, which can be best obtained by the use of concrete.

BRITISH ENGINEERING STANDARDS ASSOCIATION.

THE British Engineering Standards Association held its fifth annual general meeting recently at the Institution of Mechanical Engineers. The Chairman, Sir Archibald Denny, Bart., in reviewing the year's work, laid stress on the fact that the Association was faced with a deficit of some £1,700, and he expressed a hope that both His Majesty's Government, who represented the purchasing public, as well as industry as a whole, would assist in putting the finances of the Association on a more stable basis. The national character of the work of this organisation, devoted to simplification and the promotion of national economy in the engineering and allied industries, is too well-known to require enlarging upon ; it is not a profit-making concern, it has nothing to gain but much to give to industry generally and the country as a whole. The continually increasing amount of work which this Association has to deal with, as well as the natural progress shown in all sections, seems to necessitate more generous financial support than is given at present. The value of the labours voluntarily given by the 2,000 engineers who form this Association, which is doing so much both for the home as well as the export trade of the country, is not generally realised.

THE INCREASE IN STRENGTH OF SHORT CONCRETE COLUMNS DUE TO HELICAL REINFORCEMENT.

By **W. D. WOMERSLEY, M.A., B.Sc., Wh.Sch., A.M.I.C.E., A.M.I.M.E.**

THE formula for the calculation of the safe load that may be carried by a short column with longitudinal and lateral reinforcement is given in both the second R.I.B.A. Report and in the L.C.C. Regulations somewhat as follows:—

$$P = c\,(A + 14A_2)\left(1 + n\frac{V^1}{V}\right)$$

where P is the safe load to be carried.

c is the safe simple compressive stress in the concrete.

A is the cross-sectional area of the core of the column.

A_2 is the cross-sectional area of the vertical reinforcement.

V^1 is the volume of the lateral ties in a length of column for which

V is the volume of the core of the column.

n is a constant determined by the form and spacing of the lateral reinforcement and has a maximum value of 32.

The values given for n seem to be theoretically unsound from the following considerations:—

The limit of the helical binding becomes a steel cylinder enclosing the concrete core but the steel not taking any vertical load. Under these conditions the steel will tend to prevent the lateral expansion of the concrete, and on this account a hoop stress (f) will be set up in the steel, which in turn produces a radial compressive stress (q) in the concrete core. Since the criterion of failure for concrete is the difference of principal stresses, the following equation at once follows for the working stresses

$$p - q = c \qquad . \quad . \quad . \quad . \quad . \quad (1)$$

where p is the longitudinal compressive stress in the concrete due to the loading (that is, the working stress that may be applied to column on account of the introduction of q by the lateral binding), p is $c\left(1 + n\frac{V^1}{V}\right)$ in which it will be shown that n has a maximum value of 3 instead of 32 as usually given.

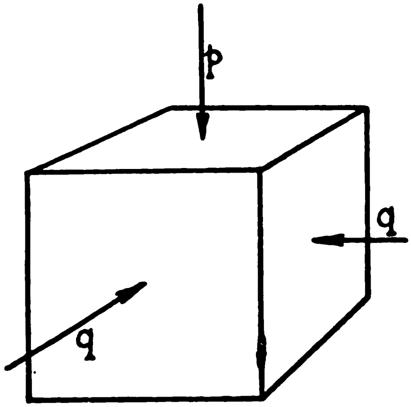
Consider the steel sheath of thickness t enclosing a concrete core of diameter D. The bursting and supporting loads on a diametral plane over a length of column (l) give the equation:—

$$q.D.l = f.2t.l$$

which may be rearranged as follows:—

$$f = q\,\frac{D.l}{2.t.l} = q\frac{4.\frac{\pi D^2}{4}.l.}{2.\pi D.t.l} = q.2\frac{V}{V^1} \qquad . \quad . \quad . \quad (2)$$

The concrete and steel must maintain contact, and hence the diametral strains in the two materials must be equal. Now, the only stress in the steel is a tension (f), and thus the diametral strain in the steel will be $\frac{f}{E_s}$, where E_s is Young's Modulus for steel.



The concrete is stressed as indicated in the diagram representing an elementary cube, and thus the diametral strain will be :—

$$-\frac{q}{E_c} + \varrho\frac{p+q}{E_c}$$

where E_c is Young's Modulus for concrete and ϱ is Poisson's Ratio for concrete

Thus the following equation results :—

$$\frac{f}{E_s} = -\frac{q}{E_c} + \varrho\frac{p+q}{E_c}$$

and if $\frac{E_s}{E_c}$ is written m

$$f = m.\varrho.p - m(1 - \varrho)q \quad \ldots\ldots\ldots \quad (3)$$

Substituting for p and f from (1) and (2)

$$q \,.\, 2\frac{V}{V^1} = m.\varrho.c - mq\,(1 - 2\varrho).$$

Whence

$$q = \frac{m.\varrho.c}{2\frac{V}{V^1} + m(1 - 2\varrho)} \quad \ldots\ldots\ldots \quad (4)$$

From this it follows that the larger ϱ the larger will be q, and hence p, for a given value of $\frac{V}{V^1}$.

ϱ was given, in the Mechanical Sciences Tripos 1912, Cambridge University, as 0·4. The Dictionary of Applied Physics, Vol. I, gives values varying from 0·08 to 0·18. The maximum possible value of ϱ may be easily shown to be ·5.

Taking m as 15 and ϱ as 0·4 in equation (4)

$$q = \frac{6cV^1}{2V + 3V^1}.$$

Now, in general, V^1 is small in comparison with V, and thus, for a more convenient result, $3V^1$ will be neglected in comparison with $2V$, and thus

$$q = 3.c.\,\frac{V^1}{V}.$$

and by substitution for q in equation (1)

$$p = c\left(1 + 3\cdot\frac{V^1}{V}\right) \quad \ldots\ldots\ldots \quad (5)$$

That is, the constant n should be 3, and not 32 as generally given. The value 32 is obtained by assuming that the lateral steel may be stressed up to its elastic limit and taking no notice whatsoever of the lateral strain condition. Thus, considering only equations (1) and (2)

$$p = c\left(1 + \frac{q}{c}\right) = c\left(1 + \frac{f}{2c} \cdot \frac{V^1}{V}\right).$$

Now, if $\frac{f}{2c}$ is to be 32 and c is not to exceed 600 lb. per square inch, f will be 38,400 lb. per square inch. This, in itself, is a high value for the elastic limit even if to allow steel to be stressed up to its elastic limit were in conformity with the usual procedure. A consideration of equations (3) and (1) gives:—

$$3q = 3{,}600 - 38{,}400$$

—that is, q would become negative if a stress higher than 3,600 lb. per square inch obtained in the steel, and it is evident that this state of affairs is quite impossible.

It follows then that for a maximum concrete working stress difference of 600 lb. per square inch the stress in the steel cannot possibly exceed 3,600 lb. per square inch, and the value of n cannot exceed 3. This makes the effect of the lateral binding practically negligible in so far as an increase in the vertical stress is considered. In columns, however, in which the load is non-axial in effect, and in which vertical steel rods must be used to take up bending moments, lateral binding must be used to prevent the vertical steel rods from buckling on the compression side. The pitch of the lateral ties need not for this purpose be less than $15d$ where d is the diameter of the vertical rods. This pitch is recommended for lateral ties, and no increase of load should be considered as being due to lateral binding.

That the ultimate load on the column will be increased is evident from a consideration of the events leading up to rupture. The concrete fails first across planes approximately at 45 deg. to the axis of the column. But the concrete is prevented from actually moving by the binding effect of the steel helix, the lateral stiffness of the vertical rods which will be considerable in the vicinity of the rupture and by the friction between the ruptured surfaces of the concrete. The column will not completely disintegrate until one or other of these three resisting forces entirely fails.

Repairs to Concrete Ships.

On July 16 last the Danish ferro-concrete steamer *Bartels*, a vessel of 1249 tons gross, struck a submerged object and subsequently grounded in Holbæk Fjord. She was afterwards docked at Copenhagen for repairs. We now learn from a firm at the latter port, states "Lloyds List," that she was found to be damaged in a furrow 26 ft. in length, with a hole in the bottom measuring 8ft. by 3 ft. The cost of repairs amounted to 1200k., or, say, £50 at the current rate of exchange. "In our opinion," our correspondents add, "a steel-built steamer under same conditions would have suffered damage which could not have been repaired for less than five or six times this amount."

SOME RECENT CONCRETE WORK IN CHILE.

(*Contributed.*)

CONCRETE construction can never become as nearly an exact science as steel, from the very nature of the material itself, and more especially the quality of the concrete is always dependent on the skill with which the work in the field is done. The details given herewith are intended to illustrate how a few little difficulties have been overcome, or how a certain design of concreting plant helped to do the work successfully. The design of the concreting plant is, perhaps, the most difficult and, at the same time, most important item about any concrete job. The whole gain or loss to the contractor may, and generally does, lie in that one item. For that reason it is hoped these illustrations may be of some assistance to engineers and contractors, even though their jobs may be entirely different in general design.

On the San Pedro Reservoir Mr. R. Bruce Tinsley was resident engineer in charge of all construction; Mr. G. H. Bayles was office engineer and made the design of the concreting plant and Mr. O. H. Pritchett was superintendent of construction. On the small tank construction Mr. H. Harvey was resident engineer in charge of all construction; Mr. G. H. Bayles was assistant resident engineer in charge of all work under Mr. Harvey; Mr. J. C. Balcomb was division engineer in charge of this part of the work; and Mr. O. H. Pritchett was superintendent. The same organization had charge of the foundry and sub-station extension. On the leaching vats construction Mr. W. Leigh was division engineer and Mr. R. C. Cormack superintendent.

SAN PEDRO RESERVOIR.

The San Pedro water reservoir is of reinforced concrete, 150 ft. sq. and 20 ft. deep. The walls are cantilever section, reinforced against the pressure of an earth bank 16 ft. high on the outside and a depth of water of 18 ft. on the inside, each without reference to the other; so that if either is removed the wall is still safe against the overturning moment of the other. This form of design necessitates a very wide base—in this case nearly seven-eighths of the

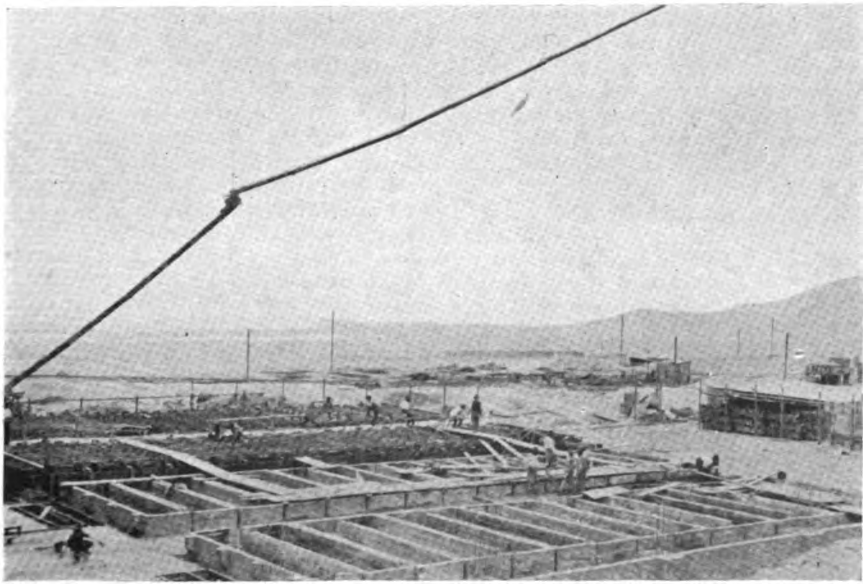


FIG. 1.—SIXTY-FOUR CONCRETE TANKS.

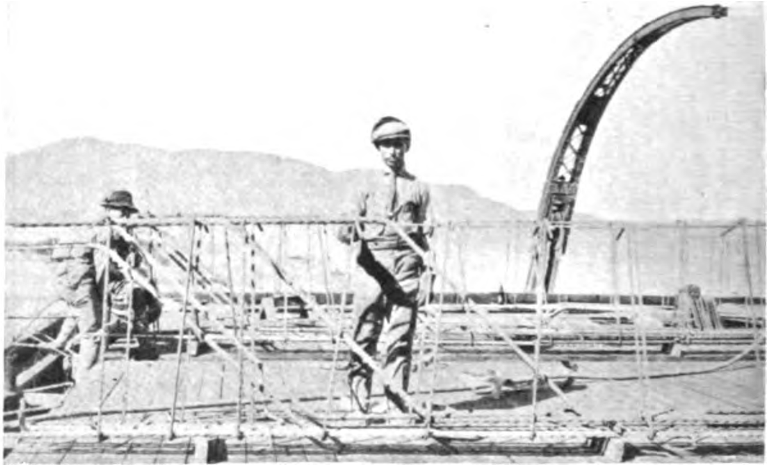


FIG. 2.—REINFORCEMENT FOR FOUNDRY ROOF. BEAM READY FOR LOWERING INTO FORMS.

height of the wall. The site was chosen at the fork of a hollow and excavation carried down to such a depth that almost the entire floor of the reservoir would rest on solid ground. A broken stone blind drain with tile near the outlet was laid all the way around under the floor just where it joins the wall base. The wall is 9 in. thick below the coping and 2 ft. 2 in. thick above the bevel at the foot. The floor is 4 in. thick, reinforced against temperature stresses only. The principal reinforcement is ¾-in. round smooth steel bars. All bar ends were hooked except top ends in the walls, where the strain is taken up by other bars or has entirely run out.

The forms were built in 6 ft. lifts, and carpenters and concreters took turns on the job. The horizontal joint (no vertical joints were made) was cleaned of laitance soon after concreting stopped and while the surface was still soft; and before concreting was resumed this joint or surface was thoroughly wetted with water and then covered with a thin coating of 1 to 1 cement-and-sand mortar. No leaking occurred on these joints.

The concreting plant was rather elaborate, considering the yardage (about 1360 cubic yards), but proved economical and satisfactory. Before any concreting was done a trestle was built around the reservoir just outside the wall line, and a 24-in. gauge industrial track laid on it. This trestle was 25 ft. high, thus reaching 5 ft. above the top of the reservoir wall and giving proper height for spouting the concrete into the forms. Wooden chutes, long enough to reach into the first lift of forms, were made and hung from the edge of the trestle, and were moved along from place to place, a few feet at a time, as the concreting progressed. As each lift was completed these chutes were cut off and served for the next higher lift. Two trucks were fitted with hopper bodies with a capacity of ½ cu. yd. wet concrete each. On a high bank above this trestle a ½-cu. yd. batch concrete mixer was mounted on a platform which had a batch-capacity hopper extending down from the top floor to the mixer. Cement and coarse aggregate were delivered alongside the railroad about 100 yds. from the mixer—the cement on a platform at car floor height, the coarse aggregate under the platform down the slope of a low fill. A trestle was built and track laid from this material dump to the mixer platform, the track branching near the dump so that one car could be loading while the other was being hauled up, dumped, and returned. The car was run under the cement platform and a charge of 3 cu. ft. of cement dumped down a chute into it. Coarse aggregate was then shovelled into it until it was filled to a mark giving 12 cu. ft. The materials track had a rise of 50 ft. in 300 ft. and an electric drum hoist was installed at the mixer platform to haul up the loads. The weight of the car was sufficient to haul the tail-rope back for the next car.

After the walls were completed the concrete for the floor was hauled in the hopper cars to convenient places and chuted down to a portable hopper mounted on a frame, from which it was wheeled to the job. Great care was taken with the puddling in the walls to ensure as nearly uniform a mass as possible. The floor was carefully screeded to proper thickness, but no particular pains were taken to get a smooth surface.

This was the first important reinforced concrete job built of ore tailings, and the experience there gained showed that the material was satisfactory if it were washed and screened and reproportioned; the extreme fines tended to weaken the mortar, and the irregularity of the sizes caused some irregularity in the completed concrete. It was therefore adopted for all concrete work, and a plant designed and built in which the tailings were screened through ⅜-in. revolving screens, thoroughly washed, and thereafter designated as sand and stone. This gave proportions of practically 1 cement, 3 sand, and 3 stone, producing a concrete that required 1·65 cu. ft. of cement per cu. yd.

SMALL TANK CONCRETING.

An unusually good example of the value of constantly repeated action in obtaining good results on construction work was on what was known as the "Tank House Extensions." These extensions are more than 416 ft. long and 169 ft. wide. The excavation covered the whole area. A floor of concrete 4 in. to 6 in. thick was laid over the ground with special reinforcement in the floor slab under the footings of the tank support columns, and with special footings for the platform support columns and building columns. The laying of the floor, building the column footings and the high retaining wall along one side, the platform which served for walkway and industrial railroad support, the platform and tank support columns, and additional regular concrete work were all more or less usual.

But 228 small concrete tanks (20 ft. long, 6 ft. deep and 5 ft. wide) were to be built, all practically one form, varying only in the end tank of each row of 16 having lugs built out at the top to support electric bus-bars.

The steel reinforcement was not very complicated, but, the tank walls being only 5 in. thick, great care was necessary both in bending and placing the steel. All the formwork was erected before any of the concreting was started. The outside forms were erected complete; the reinforcement placed, and then the

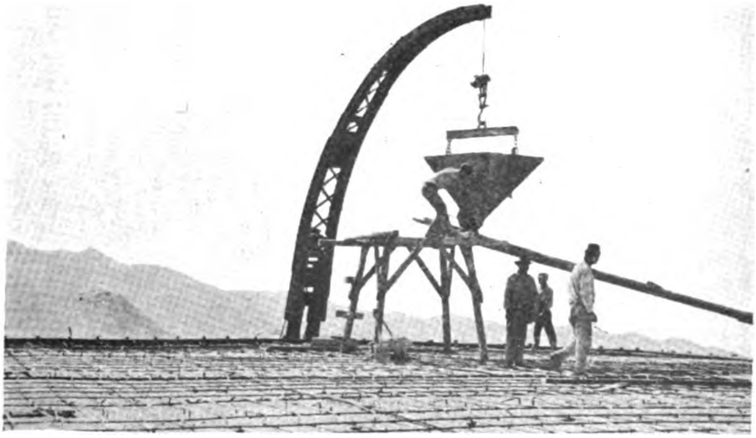


Fig. 3.—Connecting Foundry Roof.

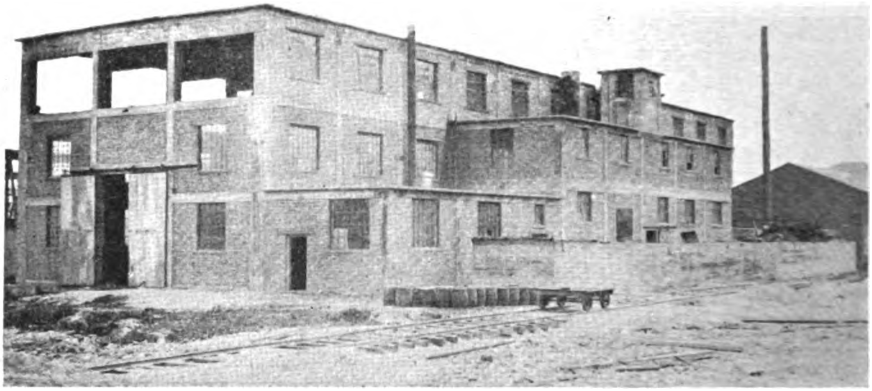


Fig. 4.—General View of Foundry after Completion.

inside form put in place. It was thought best to make the tank in one pour, and for that reason the inside form had to be supported from the top and concrete placed evenly on all sides. The bracing inside left little room for concrete finishers.

The tanks are designed with all inside corners bevelled 5 in. on the square, and the bevel along the sides at the bottom is fairly well filled with small steel bars ¾ in. from the surface and extending between other bars similarly spaced into the sides and bottom. The inside forms were removed next day, and the voids were slushed with neat cement grout and nearly dry concrete rammed into them until they were well filled. The surface was trowelled over.

CONCRETING A FOUNDRY BUILDING.

A year or two ago a larger foundry became an urgent necessity, and it was decided that the quickest and most desirable material for quick construction was reinforced concrete. The main casting room was designed to be 50 ft. wide and 180 ft. long; and in addition to this room there was an extension on the north side 20 ft. wide and 120 ft. long, 45 ft. of the length being occupied by office, toilet and tool rooms and the remaining 75 ft. being two stories high for machinery, elevator, cupolas, brass furnace, etc.; separate from the building and a few feet from it on the north side a row of 12 ft. by 10 ft. by 10 ft. storage bins for coke, iron, brass, sand, etc., was built. The core oven was also a separate structure. The building is more than 30 ft. high, and as a chuting plant was not available a locomotive crane was utilized for pouring the concrete. A standard gauge (4 ft. 8½ in.) railroad track, which became a permanent part of the work, was laid on the centre line of the foundry building. The mixer—a self-contained plant mounted on a frame that could be handled from place to place on a flat car with the aid of a crane in loading and unloading—was placed outside of and near the east end of the building. Cement, sand, and stone were unloaded beside the track as near the mixer as possible. A wooden frame containing a half-yard hopper-bottom bucket was set in front of the discharge end of the mixer. Then the locomotive crane completed the plant. This was a 20-ton crane with a special "gooseneck" extension which would just conveniently and safely handle the weight of the half yard of concrete and the bucket. Sometimes, when concreting on the roof out of reach of the crane boom, an extra chute improvised from old corrugated iron sheeting was used, one end resting on a low wooden horse and the other end resting on the concrete or reinforcement.

The mixed batch was dumped into the

bucket and immediately hoisted by the locomotive crane and transported to the job. The crane revolved and advanced as it lifted so that almost by the time the bucket was high enough to clear the roof forms the machine had reached its destination and the concrete was poured out of the bucket into the forms. In the foundation work and most of the roof concreting the whole batch could be dumped at once; but in the column work and wall beams the flow from the bucket was controlled by the gate on the bottom of the bucket so that proper opportunity should be given the men to spade or puddle the concrete in the forms.

The wall beams were formed in one piece; the columns, in sections corresponding to the heights between wall and crane beams; the roof beams in three sections of 16 ft. each besides the connections to the column forms. The first lift of the columns was concreted, and then the forms were lifted by the locomotive crane. The crane also handled the 6 in. by 8 in. timbers and 6 in. by 6 in. posts used to support the roof beam and slab forms, and later the beam and slab forms themselves. After the forms had been placed the crane carried and placed the steel used in reinforcing the concrete of the roof. The column reinforcing was built up on the ground, firmly tied together so it could be handled, and then lifted with the crane and set in place.

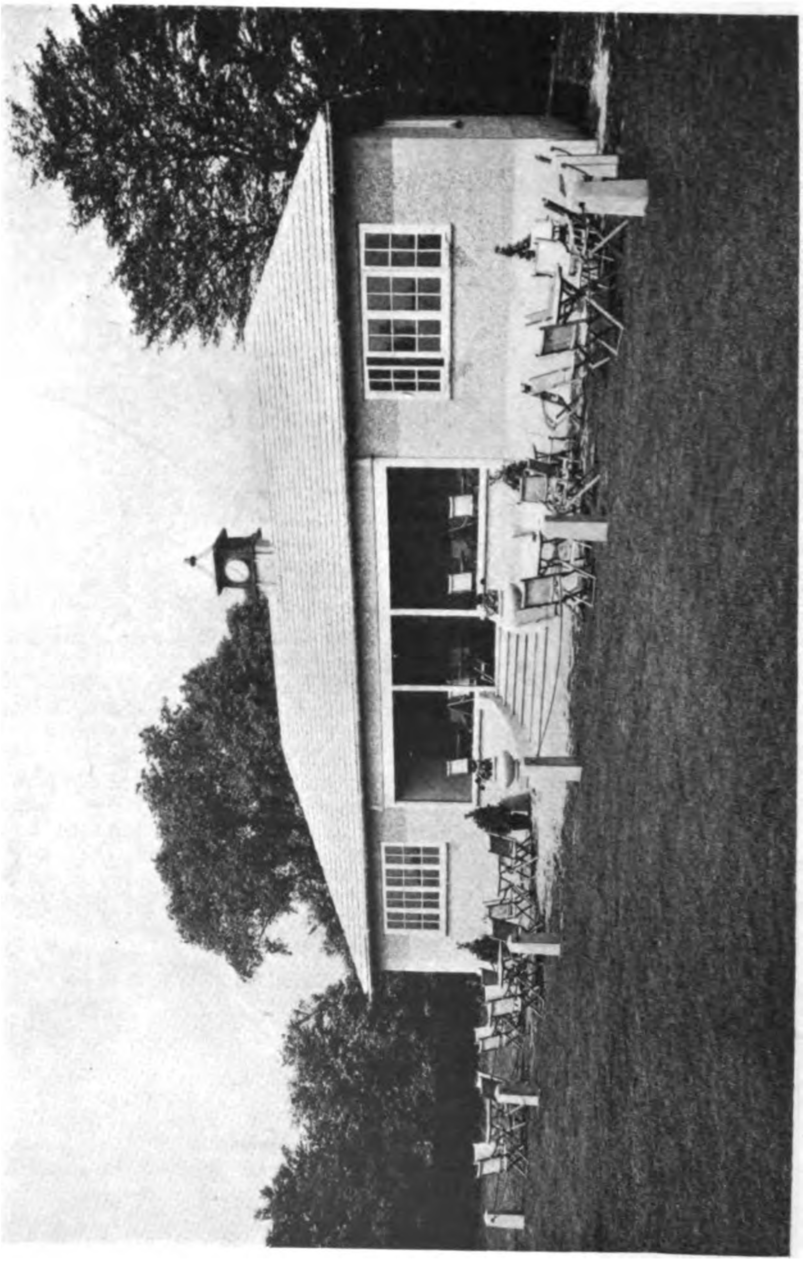
Complete forms for three bays only were built and as soon as the concrete in one three-bay section was sufficiently set to permit it they were taken down and set up for the next section. Two days were allowed for columns to set, one week for roof slabs, and ten days for roof beams. Posts were set up under the third points of the roof beams for an additional ten days. The roof sag under these conditions was less than two-thirds of that calculated, using the ordinary assumptions for strength of concrete.

LARGE VATS.

The largest single concrete job at Chuquicamata was the construction of a set of three vats in one single piece, the walls and bottom being continuous and the vats separated by division walls. Each vat is 110 ft. by 150 ft. inside and 19 ft. 6 in. deep. The floor is 11 in. thick; the walls 9 in. at the top and 2 ft. 6 in. at the bottom; the completed structure is 151 ft. 6 in. wide, 332 ft. 3 in. long, and 20 ft. 5 in. high outside. The supports between piers and columns were 5 ft. 9 in. square at the top, spaced 10 ft. on centres both ways, and placed on massive footings. The load carried on the floor is 2,600 lb. per sq. ft. The total amount of concrete in the structure is more than 12,500 cu. yds. The reinforcement in the floors of these vats was plentiful and well secured into position, so that it was possible to haul the concrete into position without any displacement. The whole of the 12,500 cu. yds. of concrete were completed without a flaw, and the vats were submitted to a water test, but showed no sign of leakage.

CONCRETE IN MUNICIPAL WORK.

In the course of a paper read at the recent annual Congress of the Royal Sanitary Institute at Hull, Mr. G. W. Palethorpe, A.M.Inst.C.E., dealt with the many uses of concrete in the constructional works of local authorities. After outlining the advantages of concrete for this purpose, he said a very great advantage at the present time from a municipal point of view was that the work could be used for unemployment relief works, and grants could be obtained from the Government for that purpose. Many bridges had been constructed recently by the aid of these grants that would probably have had to wait for ten years or more, and with the great increase of motor traffic in recent years and the very weak condition of many highway bridges all over the country a great saving had been effected for the ratepayers. There was no other material which could be used for building purposes that could be carried out by " unemployed " labour.



[Messrs. Henry Tanner, FF.R.I.B.A., Architects.

ALL-CONCRETE SPORTS PAVILION AT SNARESBROOK. (See p. 577.)

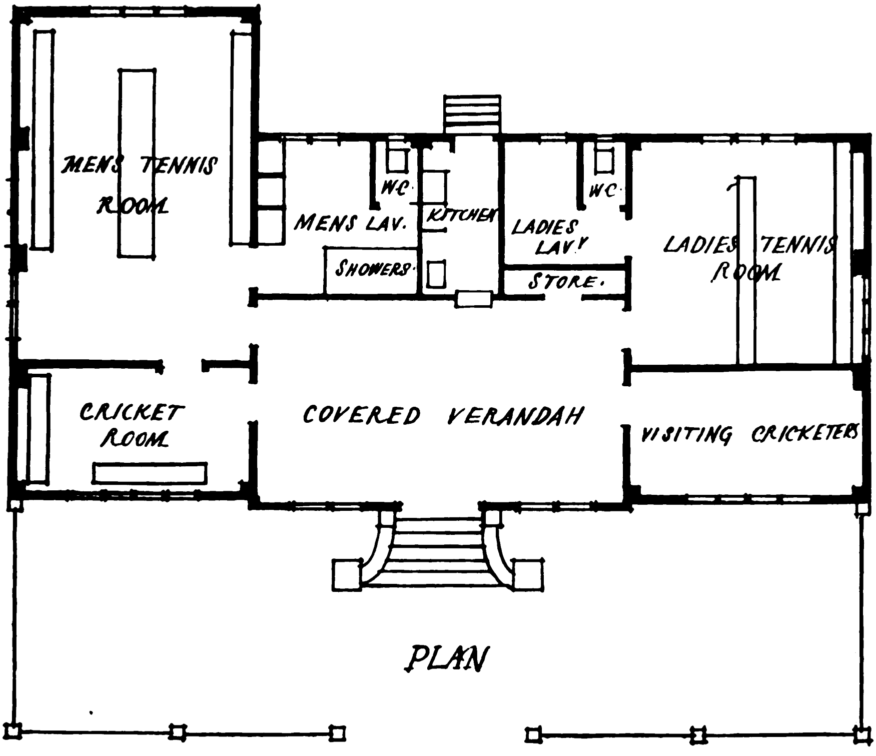
AN ALL-CONCRETE SPORTS PAVILION.

THE sports pavilion illustrated herewith (and also referred to in our Editorial pages) which has just been completed for the Associated Portland Cement Manufacturers, Ltd., is situated in pleasant forest surroundings at Snaresbrook. The entrance is from Woodford-road and the new pavilion has been built at the north-west corner where it commands a view of the cricket field and the tennis courts. It has a simple and well-balanced elevation, with a broad stairway approach to a loggia, or lounge, which provides a protected view of the grounds. A feature is made of a simply designed clock turret, placed centrally on the ridge, and visible from all parts of the ground. Appropriate to the chief interest of the company, cement products have been largely used in the construction of the building.

Accommodation is provided for about eighty members, and includes dressing rooms for men and women. Lavatory accommodation with showers for the men is provided, and a small service kitchen with gas and hot-water supply is arranged with a service door communicating with the verandah, where refreshments are served.

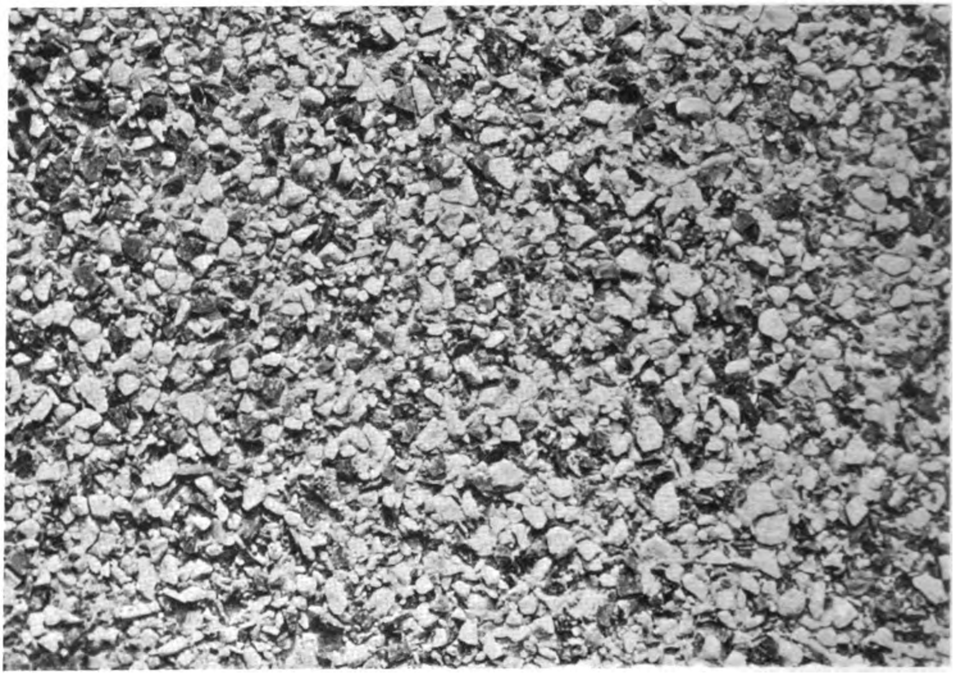
The walls are built of hollow cement-concrete blocks roughcast outside and rendered smooth internally. Grey concrete interlocking tiles have been used in the roofing, and asbestos rainwater goods are provided.

The verandah, which is approached by concrete steps, is arranged with removable glazed sashes and doors enabling the space to be used for bridge parties, and a suitable area in the foreground is enclosed with a concrete post



[Messrs. Henry Tanner, FF.R.I.B.A., Architects.

ALL-CONCRETE SPORTS PAVILION AT SNARESBROOK: PLAN.



SURFACE FINISH OF WALLING (BLACK AND WHITE MARBLE CHIPPINGS), ONE-THIRD ACTUAL SIZE.

and chain railing. Well-shaped vases, of moulded concrete, are placed at each side of the foot of the steps.

The ceilings throughout are of asbestos sheet panelled out with strips of the same material.

The external rendering to the upper part of the walls is composed of white and black marble chippings and Portland cement in the proportion of 2 : 1. The marble chippings were graded into three sizes, namely, No. 0, ⅛ in.; No. 1, ¼ in.; No. 2, ⅜ in. to 7⁄16 in., and mixed in the following proportions :—

Per cent.		Size.
34	White marble chippings . .	2
22	,, ,, ,, . . .	1
11	,, ,, ,, . . .	0
17	Black ,, ,, . . .	2
8	,, ,, ,, . . .	1
8	,, ,, ,, . . .	0
100		

The specification for the rendering was as follows :—" Score the breeze blocks and wet them. Put on a ¼-in. rendering of 2 : 1 cement and sand, and after this has stiffened sufficiently put on another ¼-in. rendering of marble chips in the proportion of 2 parts chips to 1 of cement, and press into first rendering. Twenty-four hours later scrub the cement off the surface, exposing the marble.

On this page we reproduce a photograph showing this rendering at one-third actual size, but this hardly does justice to the pleasant effect which the gradations of the two colours and the cement conveys when seen on the building. So far as we know this is the only structure in which coloured aggregates have been used in this country, and we understand that the Associated Portland Cement Manufacturers, Ltd., would be pleased to arrange for anyone interested to visit the pavilion.

The pavilion has been constructed to designs prepared by Messrs. Henry Tanner, FF.R.I.B.A., of 3, Hanover Square, W.1, and the contractors for the work are Messrs. Holland & Hannen and Cubitts, Ltd., of Grays Inn Road, W.C. The marble chippings for the surface rendering were supplied by Messrs. Diespeker & Co., Ltd., of 60, Holborn Viaduct, E.C.1. The tiles were supplied by the Redhill Concrete Tile Co., and rainwater goods by Bell's United Asbestos Co.

A COMBINED CHIMNEY AND WATER TANK.

THE chimney, with a water-tank, shown in *Fig.* 1 was recently built to replace one at Remiremont, Vosges, which was destroyed in the war. It was built of porous concrete blocks of special shape (*Fig.* 2), filled with reinforced concrete. The blocks were made on the site, hand-moulding being found to be the cheapest method, and as soon as they were ready the building was commenced. The latter (*Fig.* 3) involved no special difficulties, two courses of blocks being laid and then filled with concrete, which was afterwards well tamped.

The various details were designed in accordance with the French Regulations, which permit a maximum compressive strength of 700 lb. per sq. in. for the concrete and a maximum tensile strength of 17,000 lb. per sq. in. for the steel reinforcement. The wind-pressure was assumed not to exceed 1,400 lb. per sq. in.

The chimney is 150 ft. high, 14 ft. wide at the base, and 7 ft. 6 in. at the top. The walls are 8 in. thick at the base and 4 in. at the top. The base (*Fig.* 4) is a concrete block which exerts a mean pressure of 42 lb. per sq. in. on the foundation.

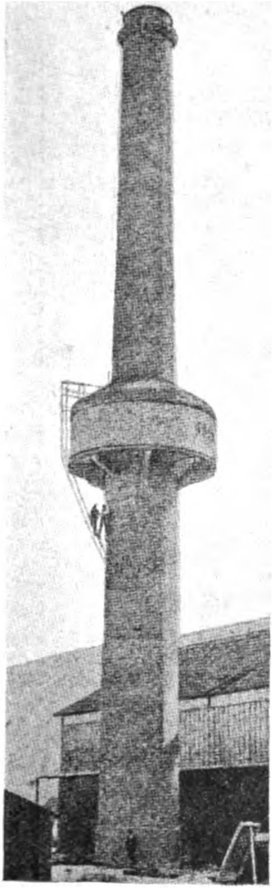


FIG. 1.

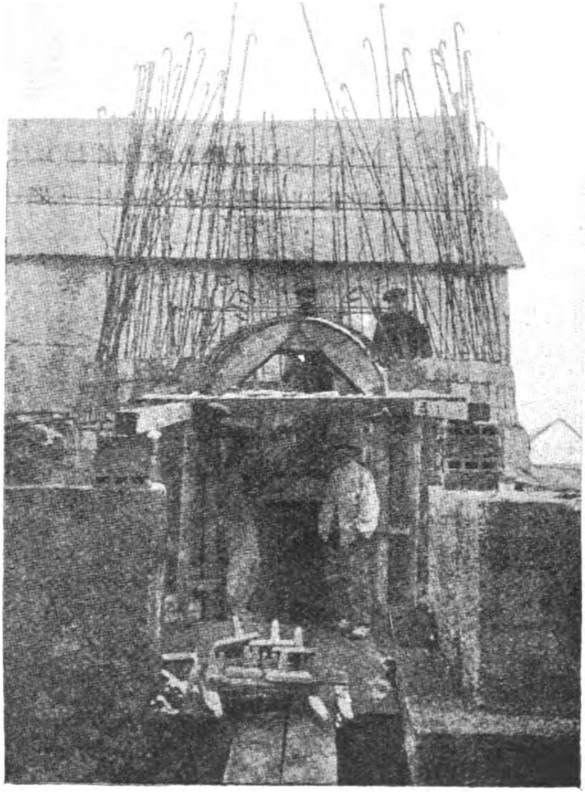
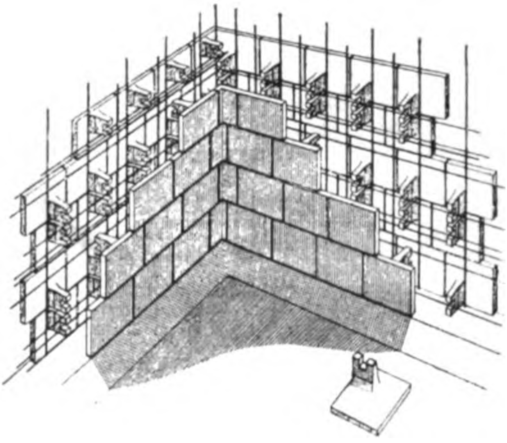


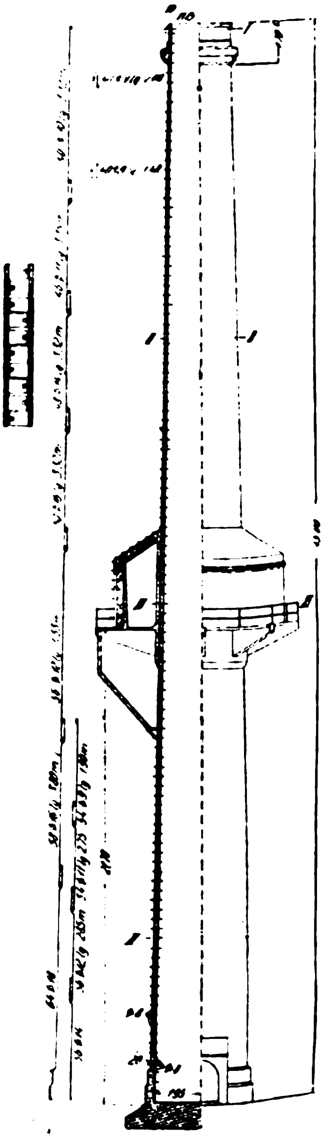
FIG. 3.

[Note: The rough texture of the chimney in the illustration is caused by the process of reproduction; in reality, the surface of the chimney is quite smooth.]

The water-tank is 70 ft. above ground-level, and has a capacity of 11,000 gallons; it was designed on customary lines and calls for no special comment.

The cost of the structure was remarkably low, viz. 45,000 francs, or approximately £1,000. The work was designed by the Mixestone Company of Geneva, and carried out by Monsieur Bodmer, the Director of the Mixestone Company in Paris.—*Beton ü Eisen.*





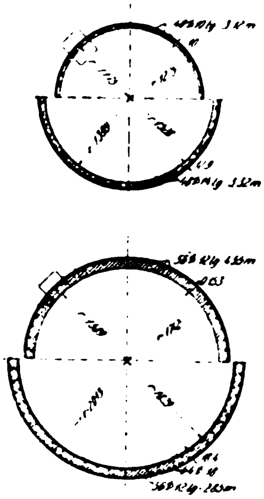


Fig. 2 (Top Left), Fig. 6 (Bottom Left), and Fig. 5 (Right).

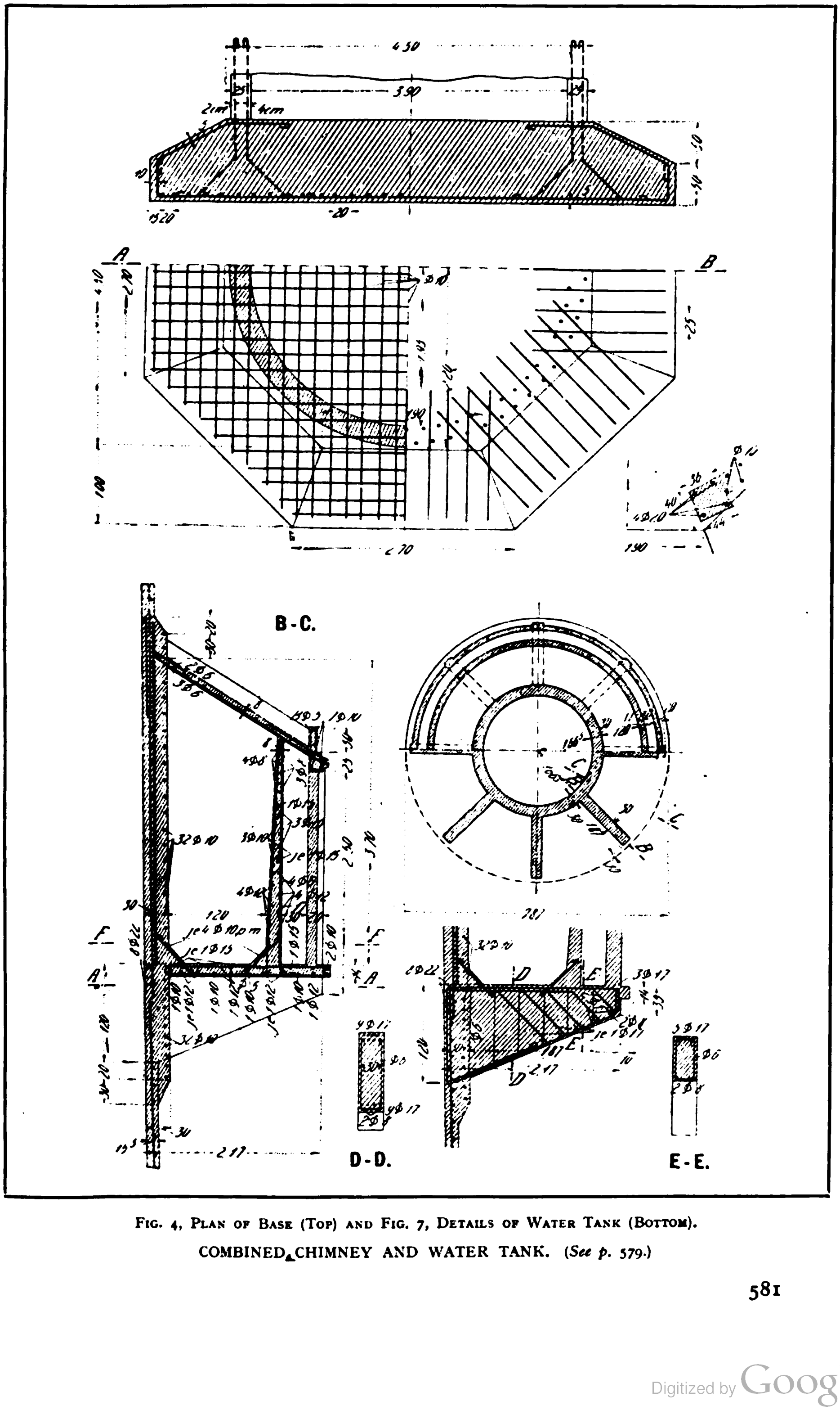


FIG. 4, PLAN OF BASE (TOP) AND FIG. 7, DETAILS OF WATER TANK (BOTTOM).

COMBINED CHIMNEY AND WATER TANK. (*See p.* 579.)

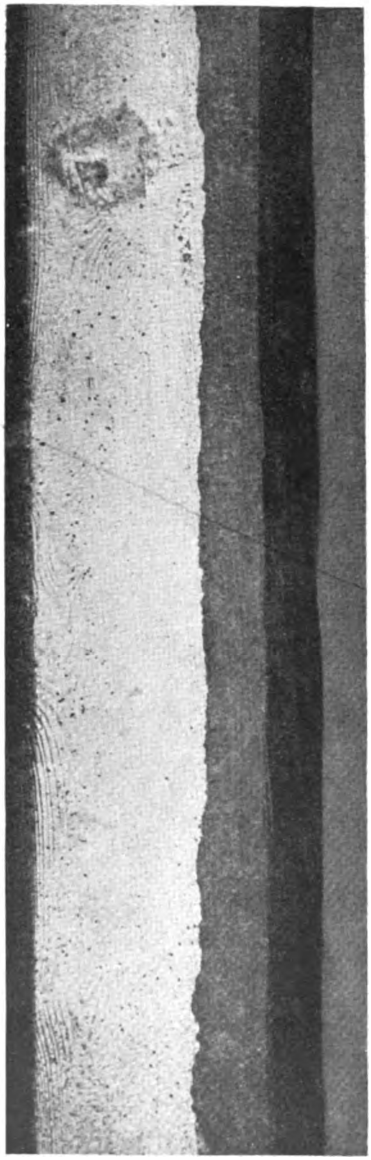


FIG. 1.—SIDE VIEW OF REINFORCED CONCRETE BEAM AFTER RESTORATION.
(The new concrete is dark coloured and the old concrete painted white.)

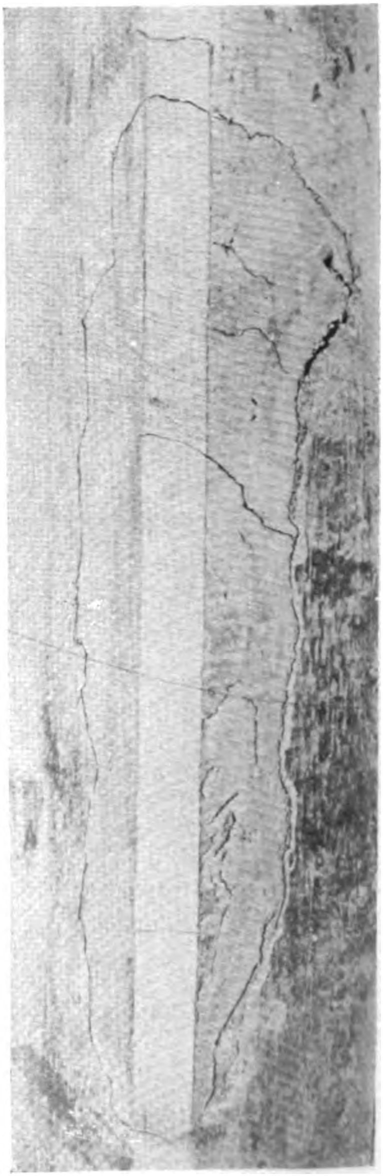


FIG. 3.—SHRINKAGE CRACKS IN CONCRETE USED IN RESTORING INJURED COLUMN.

RESTORING CONCRETE INJURED BY ELECTROLYSIS. (*See p.* 583.)

RESTORING CONCRETE INJURED BY ELECTROLYSIS.

Mr. Harold P. Brown, a Canadian consulting engineer, has made some exhaustive investigations into the prevention and repair of electrolytic injury to reinforced concrete structures, and has evolved remedies which have been found effective after long periods.

He found that reinforced concrete cracks along the lines of reinforcing rods when subjected to an electrical potential of more than 1½ volts in presence of water, even though the rods be from 5 in. to 6 in. from the surface. The cracks occur where the rods have a positive charge, and increase in width as long as the unfavourable conditions continue.

Mass concrete disintegrates in presence of water when subjected to a heavy flow of electrical current at a pressure exceeding 1½ volts. The calcium silicate and aluminium silicate are decomposed, the silicic acid appearing with oxygen at the positive pole, while the calcium hydroxide is carried to the negative pole. In all cases the presence of an acid or a salt in the water hastens the destruction.

Structural steel and foundation bolts and plates resting on brick or concrete piers are corroded in presence of water and electricity where the charge is positive. After electrolysis has parted the bolts, a continued heavy flow of electricity in the presence of leakage from gas pipes or sewers disintegrates the surrounding concrete or brick. Some kinds of limestone under the same conditions show the same results.

Unless proper precautions are taken, concrete often loses strength and crumbles when submitted to alkali-bearing waters or to sea water. The process is quickened by electricity. Damage has been caused to wharves by electricity leaking from hoisting motors and trouble may therefore be looked for where electric-motor towing-cars are operated on rails on concrete canal walls.

Brickwork, concrete, and some kinds

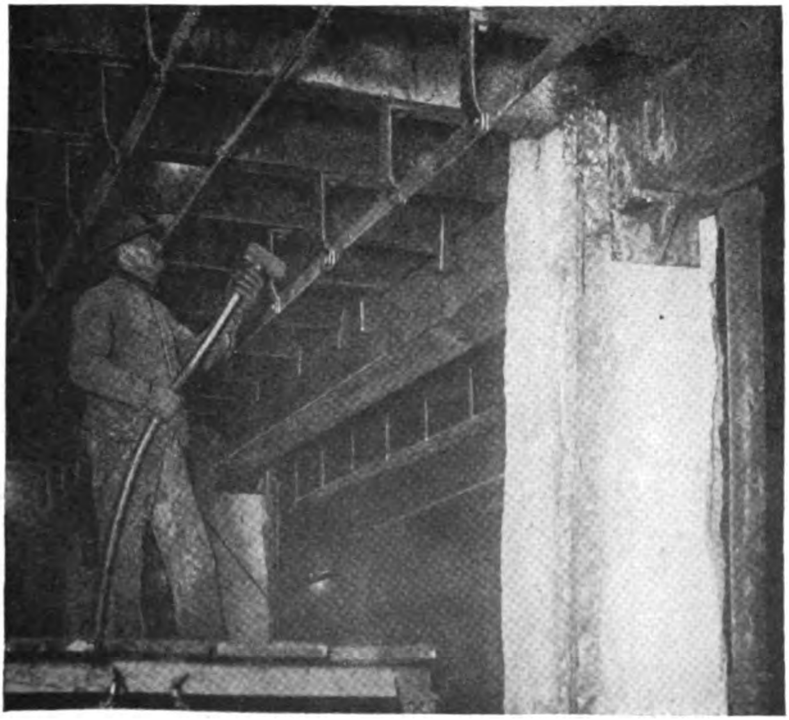


Fig. 2.—Concrete Column Injured by Electrolysis and Operator applying Concrete to Beams with Hose from Concrete Machine.

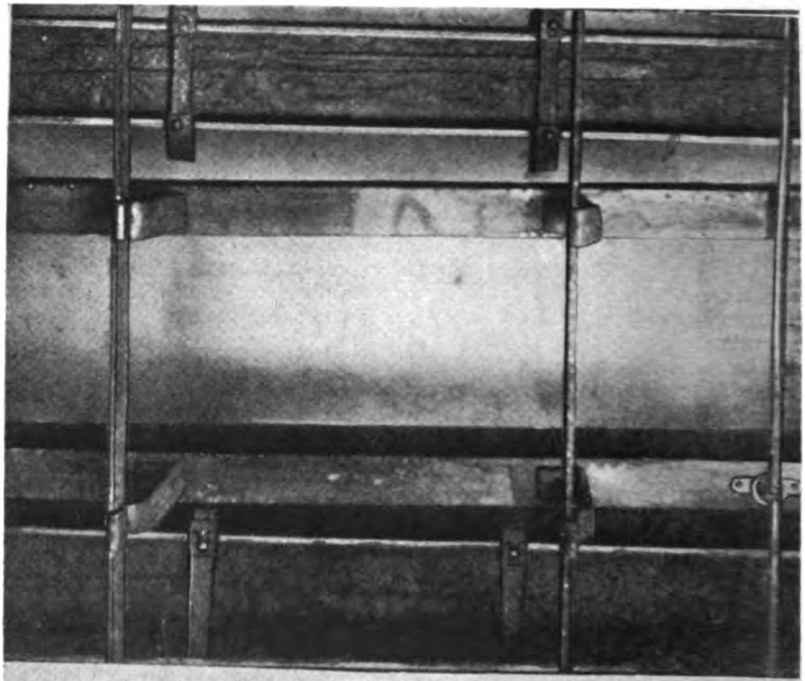


Fig. 5.—Underside of Reinforced Concrete Beams after Restoration.

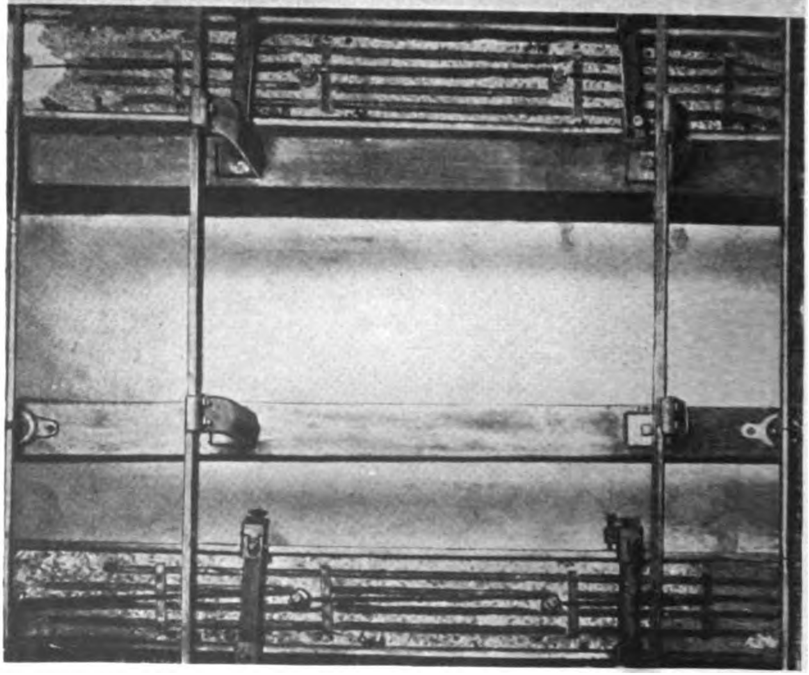


Fig. 4.—Exposed Reinforcement of Two Concrete Beams Injured by Electrolysis.

of limestone when used in long tunnels, are disintegrated when submitted to water and dense coal smoke, even when no electricity is indicated. The fact that a concrete wall is thick enough to prevent surface dampness or the visible escape of water under pressure does not prove that it is waterproof, nor does it afford any protection from trouble when submitted to a flow of electricity. Even the slight amount of moisture held on such a wall by capillary action is sufficient to afford a conducting path to the destroying current.

Permanent protection from electrolysis cannot be secured by painting or by coating a concrete structure with a membrane and waterproof material, which will sooner or later hydrolize, or crack from settlement of the structure, or disintegrate from the action of gas liquor in the water. A leak may then occur at a long distance from the spot where the waterproof layer has failed, and destruction follow in unsuspected places.

Remedies designed by Mr. Brown, have, we learn, been successfully applied in the large reinforced-concrete packing house of the Aroogast & Bastian Co. at Allentown, Penn. In this building, as the result of electrolytic action, over 3000 lin. ft. of beams, girders, and columns made of first-class concrete cracked very badly along the line of the reinforcement, and some mass concrete foundations for ammonia refrigerating coils, etc., disintegrated through the same cause.

It is stated that a large amount of mass concrete of good quality, carefully laid in 1909 and 1910 by various railways near New York, is now in excellent condition wherever it is dry, and also where it is wet without the presence of electrical current. But wherever both water and electricity have reached it the material is badly disintegrated. It is pointed out that in all of these cases the engineers spared neither pains nor expense to obtain durable work, and the cement used was of good quality.

The greater part of the injury in the concrete at the Allentown packing building was made evident by deep cracks in the concrete parallel to the reinforcing rods, which in some places were as much as 6 in. from the surface of the concrete. These cracks appeared only in portions of the building where the concrete was wet and where the reinforcement had a positive electrical charge which decomposed the moisture and released nascent oxygen on the surface of the rods. This produced a dense layer of iron oxide which required more room than the iron; consequent expansion therefore forced the concrete asunder. Wherever the concrete was dry, or wherever it was wet without a positive electrical charge, there was no damage.

It is held that the trouble in this building was caused by electricity leaking from its own power plant, passing through the broken insulation on the wires to the metal conduits containing them. These conduits were supported by cast-iron "sockets" which projected from the concrete on the lower surface of the beams and girders, forming a part of a clamping device to keep the reinforcing bars together. These sockets, which appeared every 4 ft. to 8 ft., were tapped for ¾-in. bolts and were used also to support the water, steam, ammonia and drain pipes and the overhead tramways. Steam, vapour and animal acids condensed in the conduits, which were open at each outlet. This liquid soaked through the double-braided cotton on the wires and ruined the rubber insulation, immersing the wires in a conducting fluid which, through the sockets, was in metallic contact with the reinforcing rods, thus bringing to them the destroying current.

The concrete in many parts of the building was soaked in hot water and formed a conductor of very slow resistance from the attic to the sub-basement.

The system of protection and restoration is by means of non-rusting electrical contact alloys applied to the lower surface of the "sockets" which connect all the reinforcing steel rods in each room and a testing wire carried to a circuit breaker and annunciator in the engine room. All the conduits were then suspended from the insulators, cut into sections by insulated joints, and connected to similar testing wires, etc.

Fig. 1 is a side view of a reinforced concrete beam after restoration (the new concrete being dark and the old concrete painted white). *Fig.* 2 shows a concrete column injured by electrolysis and an operator applying concrete to a beam with a hose. In *Fig.* 3 may be seen the

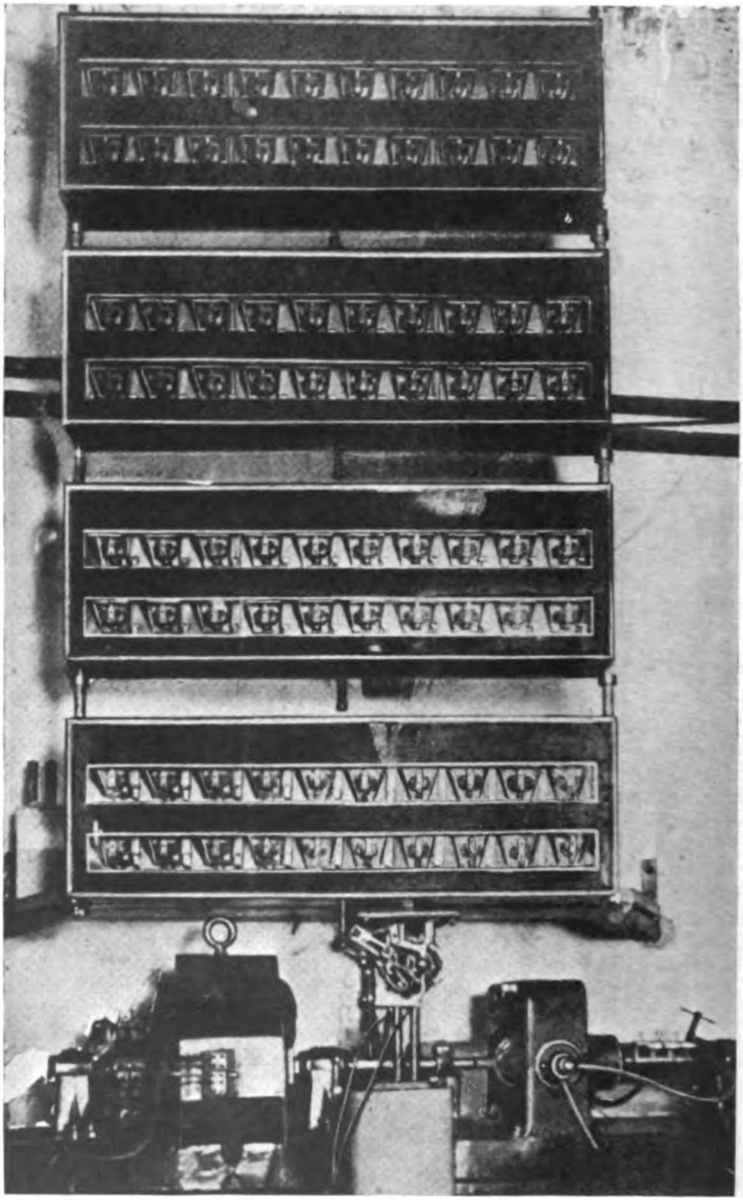


Fig. 6.—Testing Dynamo, Commutating Device, and Circuit-Breakers used for Electrolysis Prevention.

shrinkage cracks. *Fig.* 4 shows the exposed reinforcement of two concrete beams injured by electrolysis, and *Fig.* 5 shows the underside of reinforced concrete beams after restoration, while in *Fig.* 6 mày be seen the testing dynamo, commutating device, circuit-breakers and annunciators used for electrolysis production at the Allentown Packing House.

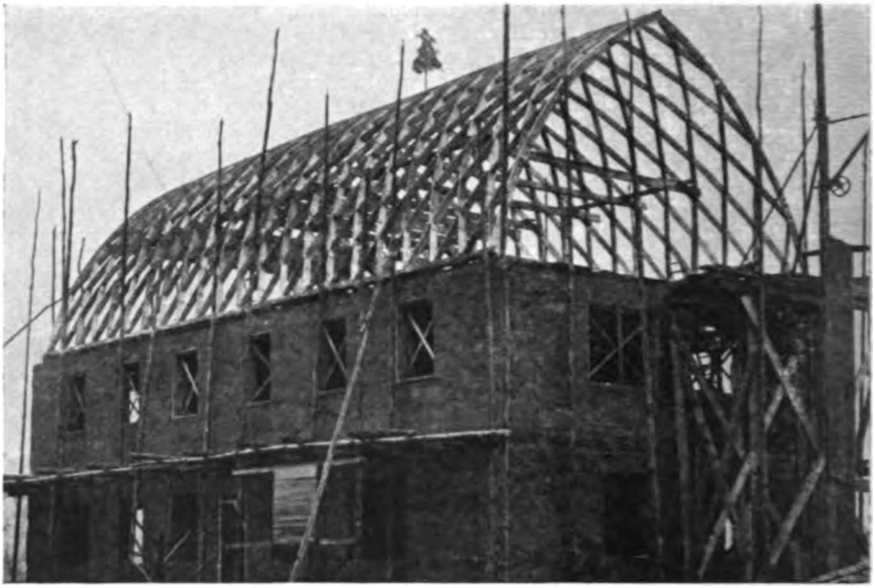
As seen in *Fig.* 6, a low-voltage testing dynamo driven by a motor is operated in the engine room, end electricity is thrown by a commutating device on to the various testing circuits. Any accidental contact between conduit metal and reinforcement operates a circuit-breaker, releases an annunciator drop, and rings an alarm bell. A similar contact between the power and light circuit and the conduit metal gives the same result. Thus two lines of defence are established, and both must fail before electricity can reach the reinforcement. The testing potential connected to the reinforcement is negative, and therefore in itself is a protection. To ensure the constant operation of the testing dynamo it is also used to supply electricity for the elevator bells.

The electric wiring conduits, outlets, and terminals, are filled or coated with a special insulating compound, which melts at about 150° F. This is done to prevent further condensation of moisture in the pipes, since parts of the building are filled with vapour. No provision is made to draw away this vapour and it is allowed to condense and freeze on the ammonia pipes in the coil loft and covers the pipes with ice, which has to be chopped off. This condensed animal vapour is an excellent conductor and is very active in corroding metal, especially when electrically charged. The switches and distribution boxes are insulated from the walls, the motors are placed on insulated supports, and all metal pipes are as far as possible insulated from the reinforcing rods. These measures have been in use for some time, and have proved effective.

A light pneumatic hammer was used to remove the cracked concrete so that the blows would not injure the solid concrete above the rods, and the thick layer of iron oxide was taken off the lower surface of the exposed rods by a sand blast. Then boards extending down to the level of the former surface were clamped on the sides of the beams and girders, and the intervening space filled with a 3-in. layer of dense, waterproof concrete which would firmly adhere to the old work. For this purpose a new apparatus was used. This machine, like a cement gun, uses compressed air to discharge a stream of concrete from the nozzle of a hose, but delivers wet concrete. In this apparatus the concrete can be properly proportioned and carefully mixed, and the hopper kept filled without stopping the work. The concrete used consisted of 1 part Portland cement to 3 parts of sand, which was firmly packed ; a float was used to level the surface, and the following morning the new concrete was found to be solidly attached to the old.

The first step in safeguarding concrete against electrolytic injury lies in the adoption of electrical devices suitable for different conditions. The next step is to make the concrete proof against both moisture and electricity—it is not enough to make it sufficiently heavy to check the flow of water ; it must be made absolutely moisture proof. In some tests it was found that properly cured concrete cubes containing 1-in. steel rods, when placed in salt water and connected to the terminals of a dynamo with a potential of 125 volts d.c., had in two days a coating of iron oxide around the positive rod dense enough to crack the concrete. Seven days later this crack had widened to $\frac{1}{16}$ in. from end to end of the block through about 2 in. of solid concrete. There was no crack around the negative rod. With alternating current both rods are corroded at about half this rate.

Various oils, asphaltes, resinates, etc., were tried for waterproofing concrete, but none was found to stand up under the test. Oils and asphaltes were found to injure the tensile strength, and sooner or later hydrolize and wash out when exposed to water and the weather. None of them would withstand the combined action of heat and water. Soap compounds, such as stearates and resinates, may at first make the concrete repel water, but they soon lose their efficiency, especially when exposed to leakage from sewers and gas pipes.



Roof from Exterior.

Roof from Interior.

CONCRETE COTTAGE CONSTRUCTION IN GERMANY (*See p.* 589.)

A GERMAN SYSTEM OF CONCRETE COTTAGE CONSTRUCTION.

By Dr. **ALFRED GRADENWITZ.**

In view of the enormous demand for dwelling-houses prevailing practically in all European countries, endeavours have been made in Germany, as elsewhere, to simplify building processes as far as possible and thus to increase their efficiency and output.

In the new method described herewith the walls of the houses are cast in a specially-designed mould which can easily be put together. The moulds are made of wood, and being composed of sections they may be made in quantities in advance of sizes and shapes to meet any possible requirements. Special moulds are provided for special types of houses, although any given moulds might, by adding or taking away some sections and their supports, be adapted for any particular type of house. The moulds are erected entirely without nails or screws, and the whole erecting process only takes a few hours. The taking down of the moulds is equally simple, and can be accomplished without damage either to the moulds or the fabric of the walls. Relying on the natural resiliency of the wood, the construction of the moulds is devised in such a way that the component parts always form close-fitting joints. The sections are kept in place securely within their frame-work by wooden cross-bars turning on a pivot. Each mould can be used for the construction of at least thirty-five houses if kept in good repair. Both the window-sashes and door-casings are fitted in with the mould and form a part of the latter, thus saving the cost of fitting in the windows and doors afterwards.

When employing about ten workmen, the moulds for a one-story house can be erected in six or seven hours, while they can be dismantled by the same number of operatives in four or five hours. The same number of workers can build one house in five days.

A special feature of this type of house is the roof, which is made up of a network of laths of equal size. The laths used in this connection can be made in quantity and easily stocked, and the same lath may be used for any kind or shape of roof, no matter how long it is or how great its span. Straight roofs are made up of plain, straight laths. The fitting together of the laths is as simple as it is ingenious. It is done by means of nuts and bolts holding the whole together; should a greater strength be required,

A CONCRETE BARN

GROUP OF CONCRETE COTTAGES IN GERMANY.

reinforcing vertical or diagonal laths may be added to the structure in order to obtain the strength as established by calculation. The curved form, on the other hand, is best suited for roofs of small houses, as the space gained can be utilised for making the rooms larger. The space encompassed by this type of roof is entirely free from struts, stays, or cross-beams, and this method of roofing is particularly well suited for covering sheds, halls, factories, barns, etc.

The building process above described has been applied to the construction of a large number of houses in several European countries, and, as may be seen by the illustrations, if placed in an appropriate setting the houses give every promise of forming charming little groups. The barn shown in one of the illustrations is particularly attractive.

IMPROVEMENTS IN SHAFT CEMENT KILNS.

MANY small improvements have been made in shaft kilns for Portland cement during the last eight years (1914-1922). These relate to various portions of the kiln rather than to any new principle.

Thus, the automatic filling of shaft kilns is ensured by the use of briquettes of cement-mix, these being delivered to the kiln at a uniform rate by means of a mechanical feed and distributed by an automatic scraper or rake which revolves around the top of the kiln. The fuel, in the form of coke breeze, is supplied in alternate layers with the briquettes. The details of the mechanical devices which control the supply of briquettes and fuel differ with each firm supplying them.

The automatic discharge of shaft kilns is effected by various devices, of which the most important are: (*a*) curved bars which, when partially rotated, form a sufficiently large opening to permit the clinker to pass between them; (*b*) a rotating base plate which discharges a definite quantity of clinker at each revolution; (*c*) a bucket conveyor which removes the clinker from below the kiln and so causes a further quantity to fall out of the kiln; and (*d*) various shovel-like devices which dig out the clinker automatically.

The air supply, both for cooling and combustion, has been placed under much better control than in the older kilns. It may be divided into three parts: (*a*) air used to cool the clinker sufficiently to enable the latter to be handled conveniently—incidentally the heat imparted to the air by the clinker facilitates the combustion of the fuel; (*b*) hot air admitted directly to the zone of combustion; and (*c*) hot air used to dry or pre-heat the cement mix. An investigation of the amount of heat evolved by the fuel, the amount of air required to cool the clinker, and the heat needed to dry and pre-heat the raw mix, will show that there is ample scope for full utilisation of all these sources of air.

The use of fan-draught instead of natural draught is also increasing, as it provides a far better control of the kiln and eliminates the variations due to climatic conditions.

Usually four or five kilns are connected to a main flue, from which the gases are exhausted by an induced fan of 40 h.p., but in some works each kiln has a separate fan. It is sometimes economical in power to use a separate fan for each kiln, as leakages in a long main are thereby minimised and if a kiln is out of use at any time no power is wasted. The draught can be regulated far better when each kiln has a separate fan.

The kinds of grates used in modern cement kilns vary from a simple grate to various complex auto-discharging grates. Sometimes the grates are vertical, with an automatic discharge below; in other kilns the grate is solely horizontal; and in still other kilns both horizontal and vertical grates are used. In a few cases an air-cone—similar to that used in some gas producers—is placed in the bottom of the kiln and the air is injected through this cone.

The methods used in preparing the raw mix and in forming the briquettes have not been improved in any striking manner during the past few years. The improvement made consists chiefly in a more thorough mixing and in the use of heat which would otherwise have been wasted. Usually, layers of coke and briquettes are used. In one type of kiln the free fuel is only burned in the outer zones; the material fed to the centre of the kiln consists of a mixture of raw meal and powdered fuel.—*Tonindustrie Zeitung*.

CORRESPONDENCE.

CONCRETE-ENCASED STEEL FLOOR JOISTS.

SIR,—In your issue for August, Mr. E. S. Andrews wrote an article on the above subject, and asked for further information.

Strength tests of concrete-encased steel beams have been carried out in America and reported in *Engineering News-Record* of May 3, 1923, and June 28, 1923 (Vol. 90, No. 26). The following summary may therefore be of value.

The results,* recently reported by Profs. H. M. Mackay, Peter Gillespie, and C. Lelnau to the steel fabricators' section of the Canadian Manufacturers Association, are held to indicate that in such encased-beam construction the concrete and steel act together to form a composite beam, which may be designed by the usual reinforced-concrete **T**-beam theory; that this action may be utilised to effect a considerable economy in floor design; and that the limits of bond stress between concrete and steel which should be observed in such design require further tests to determine them. The experimenters also report that it is advantageous to provide for continuity at the ends of the beams by reinforcement passing through the top of the slab, that the shear reinforcement provided by the webs of the beams will tend to eliminate failure from diagonal tension, and that these encased beams are in a more favourable position than reinforced-concrete beams because of the fact that the concrete carries no stress from dead-load.

The live-load was carried up to 365 lb. per sq. ft., or 3½ times the design live-load or more than twice the design total load. This increased the live-load flange stress in the steel from 7,600 lb. per sq. in. to 14,750 lb. per sq. in. and made the total flange stress (dead plus live) 21,500 lb. per sq. in. Slight haircracking in the lower portions of the beam encasement, one or two cracks extending to near the bottom of the slab, appeared during the higher stages of loading. The test was discontinued because it proved impracticable to place more load on the panel. Comparison of the measured steel stresses with those computed by calculation of the structure as a reinforced-concrete assemblage showed the observed results to lie between those calculated for the case of full tension in the concrete and the case of no tension in the concrete; in the latter case a part of the slab was counted as acting with the beam encasement to give **T**-beam action, this slab width extending four times the slab thickness to either side of the beam encasement. In the lower stages of loading the results agreed closely with those for the case of tension in the concrete, while in the upper stages of loading the results diverged more and more toward the other basis of calculation.

J. SINGLETON-GREEN.

Audenshaw,
Manchester.

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* *E.N.R.* 28–6–23. 90–26.

REINFORCED CONCRETE BRIDGE DESIGN.

SIR,—On page 247 of your issue for April, 1923, certain details of a concrete bridge at Loughor are given. From perusal of this, and of a somewhat similar structure at Taunton, described in your issue for January, 1923, it would appear that the economic factors involved are somewhat different from those under which work is at present being done in Victoria.

The bridge described has spans of 40 ft., and width of 36 ft., designed to carry 120 lb. per sq. ft., and two 24-ton traction engines, presumably placed in the least advantageous position. For this purpose four main beams 12 in. by 32 in. are provided.

A standard road bridge of this Board is designed for one 15-ton traction engine and allows a clear width of 20 ft. (without footways) carried on four beams, each (for a 40 ft. span) 12 in. by 36 in. with four 1⅛-in. bars and four 1-in. bars, where the spans are continuous over three piers. On these beams, which are spaced at 5 ft. 3 in. centres, is a 7½ in. deck without secondary beams.

The Loughor design is carried on four main beams 12 in. by 32 in., although the bridge is over twice as wide, and designed for loading about twice as severe. It is to be presumed that a considerably higher percentage of steel is used: in this case, how is the shear dealt with at, say, 5ft. from a pier, where the beam has practically no haunch?

This design allows for no expansion, save in parapets, in 720 ft. of concrete. In Victoria 200 ft. without a split expansion pier is seldom exceeded in a bridge of this type: and in these the "Tarmac" deck almost invariably cracks, showing that the joints are indubitably effective. It would be of interest to learn if such lengths without this provision are common in English practice.

It is worthy of note that at present contract prices such a bridge as that described would cost in this country about £45 per ft., as against about £80, the contract price mentioned in your article.

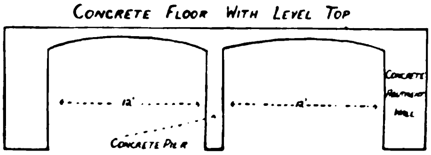
A. E. KELSO, M.C.E., A.M.I.C.E.
Bridge Engineer,
Country Roads Board,
Victoria.

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

***Readers are cordially invited to send in questions relating to concrete, which will, as far as possible, be answered direct and subsequently published where they are of general interest. We cannot give complete designs; these are the province of the consulting engineer. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.*—ED.**

REINFORCEMENT FOR FLOORS.

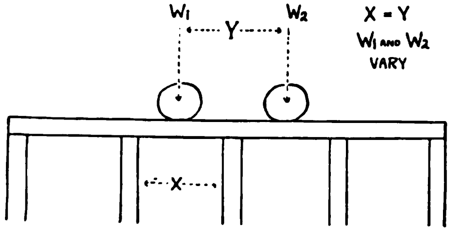
QUESTION.—How should the amount of reinforcement be calculated for an arched concrete floor (as sketch) to carry a moving load of three tons on a wheel 6 in. wide?—T. P.



ANSWER.—It is clear that the concrete abutments would not be strong enough to resist the thrust of the arch without spreading unless they have some support not indicated in the question. Under these circumstances it is best to design as a continuous beam resting on three supports. The effect of any arching which may take place would not be to reduce the thickness, but solely to reduce the amount of reinforcement required. The design as a continuous beam has already been treated several times, and presumably presents no difficulty.

FLOOR LOADS.

Question.—A beam supported on pillars carries various loads from side beams (floors), and in addition a rolling load. All loads vary at different times. In order to ensure complete stability the beam is designed as supported at ends. Owing to deflection and the beam tending to function as a continuous beam, cracks may appear over points of support. What reinforcement should be placed at these points to prevent this? *Expressed in another form.*—If a continuous beam has an excessive moment of resistance in the centre, but a low M.R. over points of support, do the points of contraflexure move to points in the beam to suit the M.R.'s, or is the reinforcement over the points of support overstressed even to failure?—B. W. S.



Answer.—A continuous beam with excessive moment of resistance in the centre and a low moment of resistance over the support does shift the points of contraflexure farther towards the supports than when the beam has constant moment of inertia, but this movement is not sufficient to prevent the support sections from being overstressed and the centre section understressed. The maximum moment which can occur over the support in cases similar to that described in the question is generally of the same order as the free moment at midspan, and greater than the moment at midspan of a continuous beam. If the end sections are overstressed, or if the end moment of inertia is much less than the centre moment of inertia, this moment will be reduced, but we should consider it dangerous to design the support section for a moment of less than two-thirds of the free moment, unless the beams are made really non-continuous, with all that this implies.

WATERPROOFING CONCRETE BLOCKS.

Question.—I should be very pleased if you can give me any advice in regard to making the facing on semi-dry concrete blocks waterproof. I notice that some of your contributors refer to hydrated lime as a waterproofer. Can I take it that hydrated lime is ordinary burnt lime slaked with water, and, if this is the case, how can I reduce it to a very fine dry powder?—J. W. E. C.

Answer.—There is some conflict of evidence as to the value of hydrated lime as a waterproofing material. Hydrated lime is merely lime which has been slaked, and it is sold as a very fine dry powder. Probably the only sound method of making concrete waterproof is to produce a dense concrete by using sufficient cement and a well-graded aggregate with just sufficient water to produce a plastic mass.

SILICATE OF SODA AND CONCRETE.

Question.—When is the best time to apply silicate of soda solution to concrete surfaces, such as concrete floors, concrete blocks, etc.? Does the hardening of concrete surfaces take place even if the solution is applied after some weeks or months? May the cement finishing coats so hardened be painted, waxed, etc.? —M. J. V.

Answer.—Silicate of soda is best applied when the slabs are well set but not hardened, say at the age of a week. The action, we believe, is the combination of silicate of soda with the free lime. After some months or weeks the free lime will take up carbonic acid, and not be so ready to take up sodium silicate. This action is very slow, however, and practically the silicate may be applied any time, but preferably when the concrete is green. There is no objection to painting, waxing, etc., after hardening.

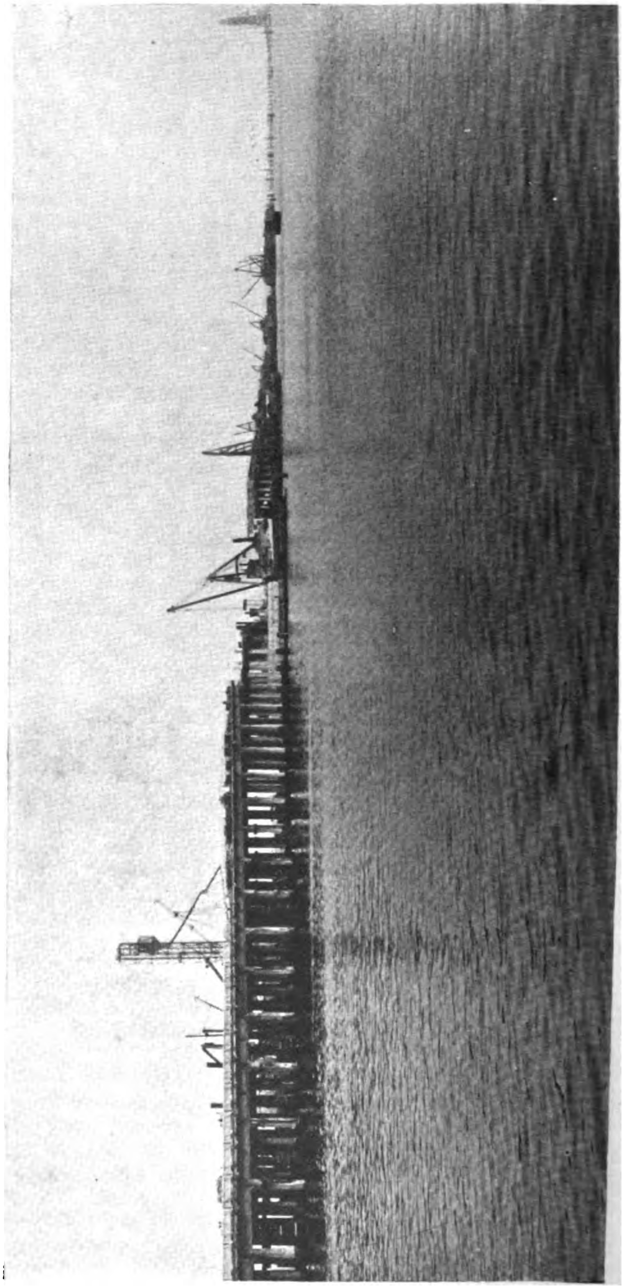


Fig. 1.—PORTION OF WOODHAVEN BOULEVARD EXTENSION, JAMAICA BAY, NEW YORK. (*See p.* 595.)

NEW METHOD OF DRIVING CONCRETE PILES.

THE illustrations given herewith show a new and economical method of driving concrete piles without the expense of stripping the heads after the piles are driven. The work in question was for the Woodhaven Boulevard Extension across Jamaica Bay, New York. This structure, which is a highway bridge, will be about 7 miles in length and crosses several islands which at high tide are nearly submerged. The full width of the bridge is to be a little over 100 ft., but at present only one-half of it is being built, that is, a width of a little over 50 ft. The bottom under water is sand with more or less clay, in some places the latter attaining a thickness of 18 to 20 ft.

Fig. 1 shows a view of one portion of the work stretched out across the Bay. *Fig.* 2 shows a few of the 24-in. square concrete piles in place. These piles vary in length from 70 to 95 ft. although some have been cast that were 105 ft. in length; they are 24-in. square practically to the point, where they are slightly tapered to receive a cast-iron shoe which is moulded into the concrete. The specifications for these piles were very stringent. The contractor was required to place the reinforcing rods not more than ¼ in. off centre as shown on the plans. The average run of the piles weighed from 25 to 30 tons each. As the reinforcing rods were so accurately spaced, the contractor took advantage of this feature and built a heavy timber template which fitted over the projecting rods and brought the timber template down directly on the concrete top of the pile. The hammer thus drove on the complete top section of the concrete pile and did not drive on the rods. This template arrangement and housing for the hammer are shown in *Figs.* 3, 4, and 5. *Fig.* 3 shows the template and framework for supporting the hammer. *Fig.* 4 shows the hammer

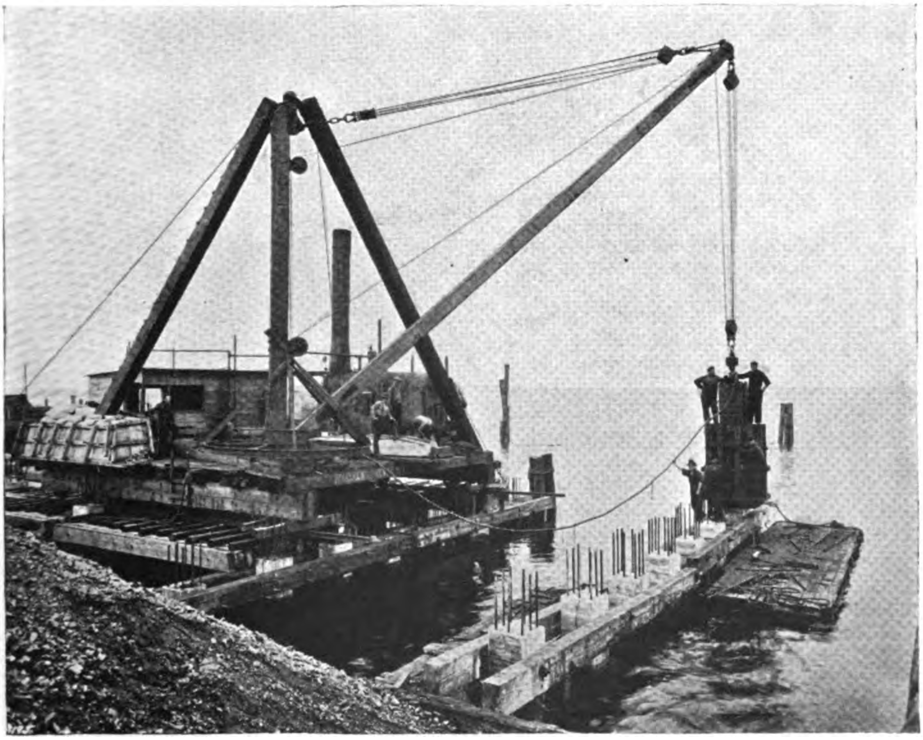


FIG. 2.—SHOWING SOME OF THE 24-IN.-SQUARE CONCRETE COLUMNS IN PLACE.

over the reinforcing rods, the latter projecting up through the template as before described. *Fig.* 5 shows the hammer after it has been set down over the reinforcing rods and is resting on the top of the concrete pile.

Our Frontispiece shows a general view of the work after some of the 24-in. piles had been driven.

The piles were first set in position and jetted down to within 4 or 5 ft. of the finished level and the jets extended about 2 ft. below the point. Through each of these four jets water was pumped to the extent of 260 gallons per minute, or over 1,000 gallons for the four jets per minute. This work was then followed by a pile driver rig as shown, with a " 11B McKiernan-Terry " automatic hammer mounted, which drove the pile to the final penetration and set.

The contractor who first handled this work attempted to set the piles by jetting, but the highway engineers were not satisfied that the penetration by jetting was the actual refusal of the pile, and had an " 11B McKiernan-Terry " hammer put on to test them out. Although a period of seven months had elapsed it was found that all the piles moved from 8 to 22 in. after one minute's driving, and in some cases more, and were finally driven home to final set with this hammer.

The hammer in question has a gross weight of 13,535 lb., with a piston weighing 3,625 lb., with a stroke of 18 in., delivering 120 blows per minute. This type of hammer takes steam on the downward as well as the upward stroke, and whilst not being suitable for driving all types of concrete piles there are many cases where it can be used economically. This size hammer was used at King George's Dock for driving two 15-in. by 15-in. timber piles, bolted together in lengths of 70 ft., through hard ballast and was successful where a 4-ton drop hammer had failed. It has also been recently used in Holland driving steel piles to a total penetration of 91 ft.

We are indebted for the photographs to the British Steel Piling Company, of Dock House, Billiter Street, London, E.C. 3, who have the sole rights of McKiernan-Terry hammer outside N. America.

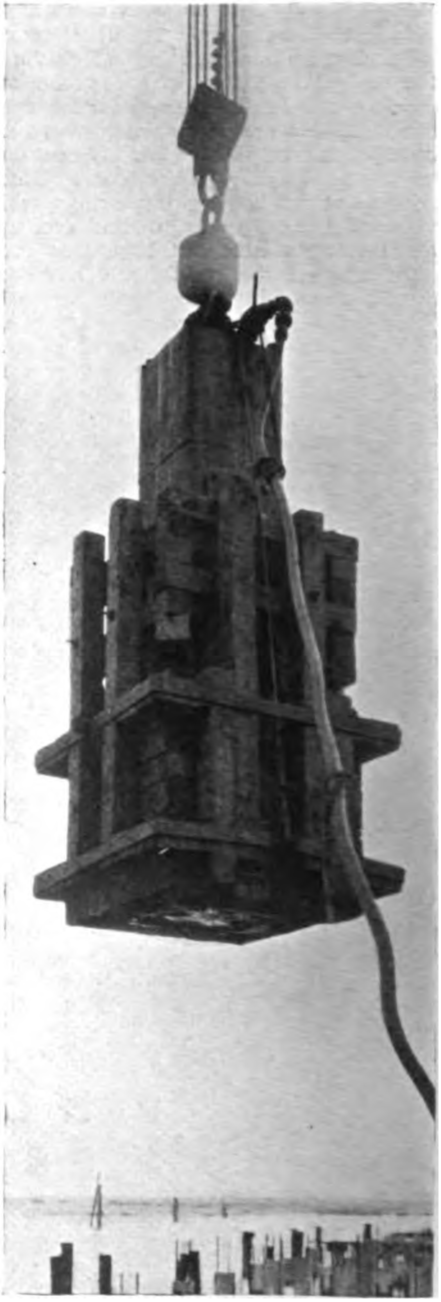


Fig. 3.

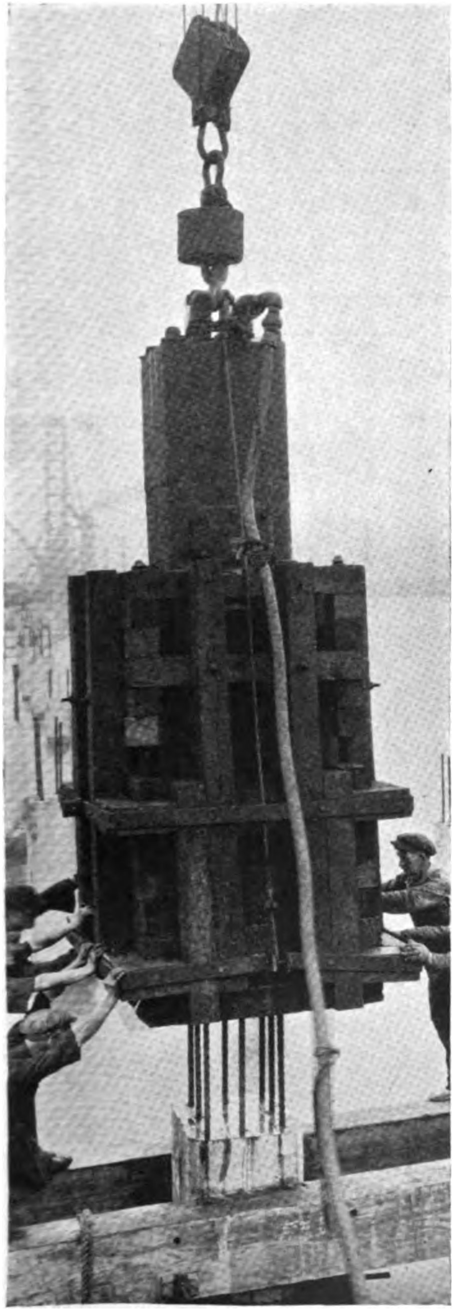


FIG 4.

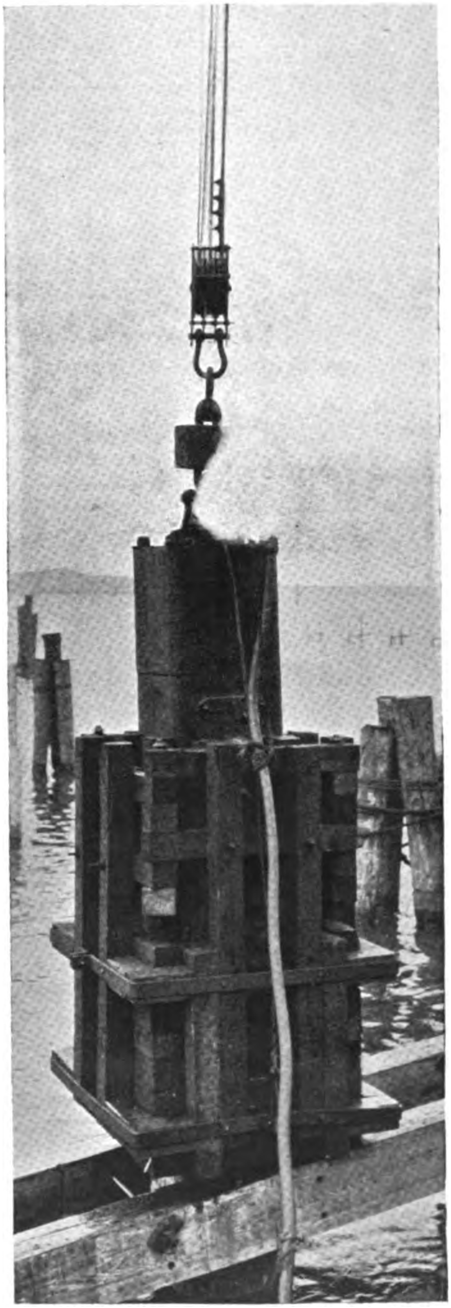


FIG. 5.

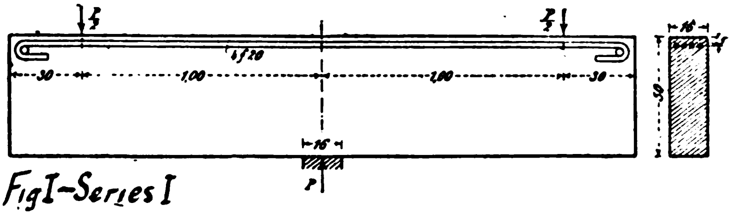


Fig I—Series I

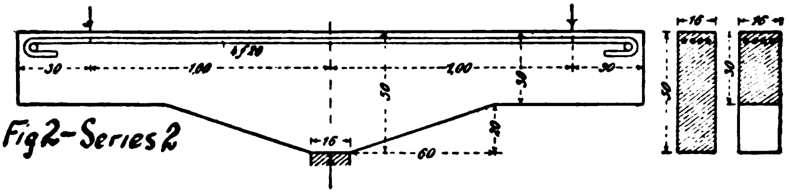


Fig 2—Series 2

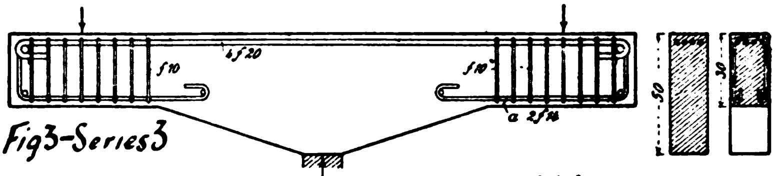


Fig 3—Series 3

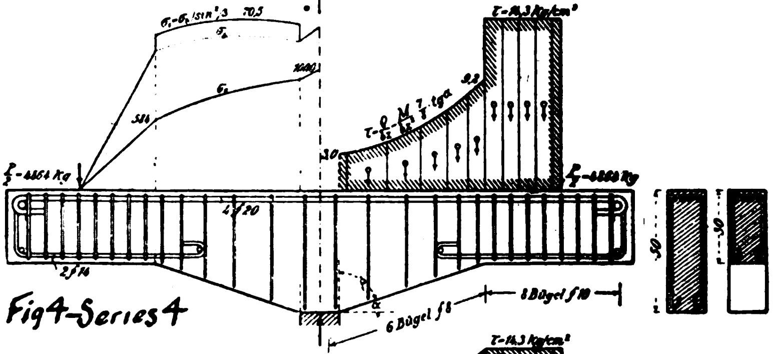


Fig 4—Series 4

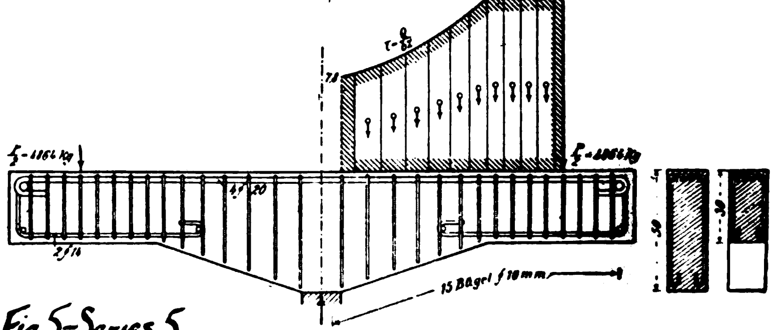


Fig. 5—Series 5

SHEAR TESTS IN ARCHED BEAMS. (*See p.* 599.)

SHEAR TESTS IN ARCHED BEAMS.

A SERIES of investigations of the shear-stresses in arched beams, recently published by Dr. E. Morsch of Stuttgart, was made to ascertain the effect of loads on the beams. Symmetrical beams with a central support were used for the tests, and they were subjected to two equal loads, placed symmetrically. The beams were also tested upside down, and also when supported on two knife-edges placed 80 in. apart and loaded centrally.

In Series 1 (*Fig.* 1) the beams were of constant section, 50 cm. by 16 cm., reinforced with four round bars of 20 mm. diameter, whose ends were bent over a cross-bar 40 mm. diameter. The load was 9,728 kg. and the shearing stress 7·8 kg./cm^2. In Series 2 (*Fig.* 2) the beams were 50 cm. high at the deepest part, but only 30 cm. at the ends. The load was again 9,728 kg. and the shearing-stress 14·3 kg./cm.^{2}, but only 9·2 kg./cm.^{2} at the beginning of the slope and only 3 kg./cm.^{2} at the point of support.

In Series 3 (*Fig.* 3) the outer part of the beams was strengthened by 10-mm. rods placed 69 mm. apart, and two rods of 14 mm. diameter (marked *a* in *Fig.* 3) were also added.

Series 4 and 5 (*Figs.* 4 and 5) differed from Series 3 by the addition of the rods extending down through the middle of the beam and in the distance between the rods. The concrete was made of 1 part of cement, 2½ parts sand, and 3½ parts gravel, with 10 per cent. of water. After 45 days in a moist atmosphere it had a crushing strength of 228 kg./cm.^{2}, and a tensile strength of 18·3 kg./cm.^{2}. The cracks in the beams are shown in *Figs.* 6–10. It is important to notice that the middle vertical cracks in the beams in Series 2 (*Fig.* 7) are far shorter than those in Series 1 (*Fig.* 6); this can only be due to the shape of the beam. The effect of the shape of the beam is still further shown in Series 3 (*Fig.* 8), especially when the much greater loads are considered.

Series 4 and 5 (*Figs.* 9 and 10) do not differ materially from each other.

The tests show that the formula

$$t_o = \frac{Q}{b_o Z} - \frac{M}{b_o Z^2} \cdot \frac{7}{8}\, tg\,a$$

devised by the author in 1908 is a perfectly safe one, if t_o is the shear, Q is the maximum load, M the moment, and the other symbols have their usual significance.—*Beton u Eisen.*

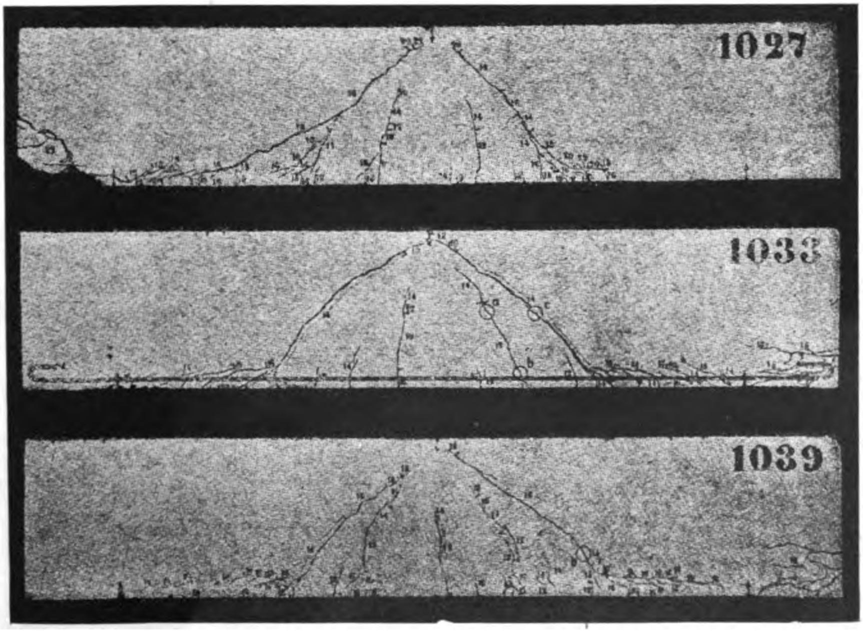


FIG. 6. SERIES 1 (MAXIMUM LOADS 22, 18, AND 18 TONS RESPECTIVELY).

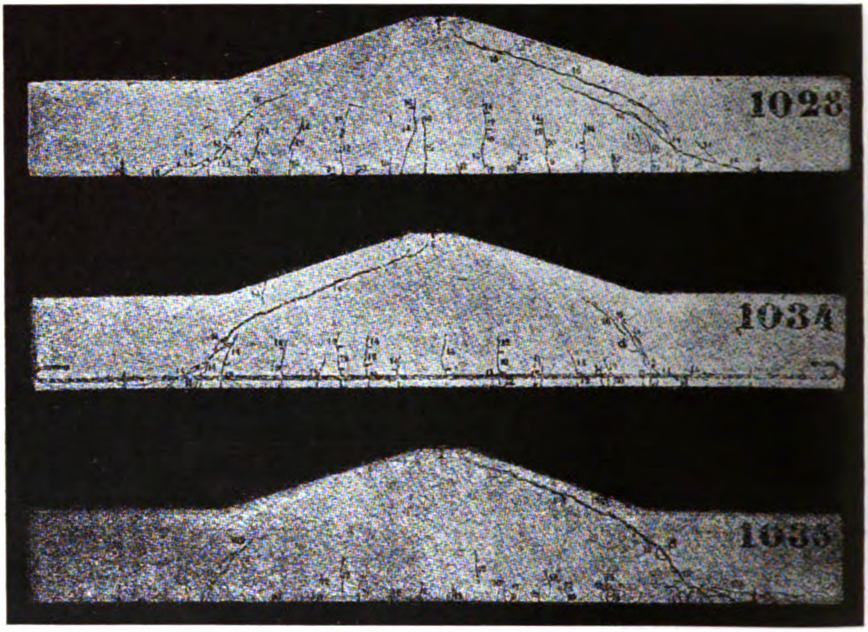


Fig. 7. Series 2 (Maximum Loads 16, 16, and 14 tons respectively).

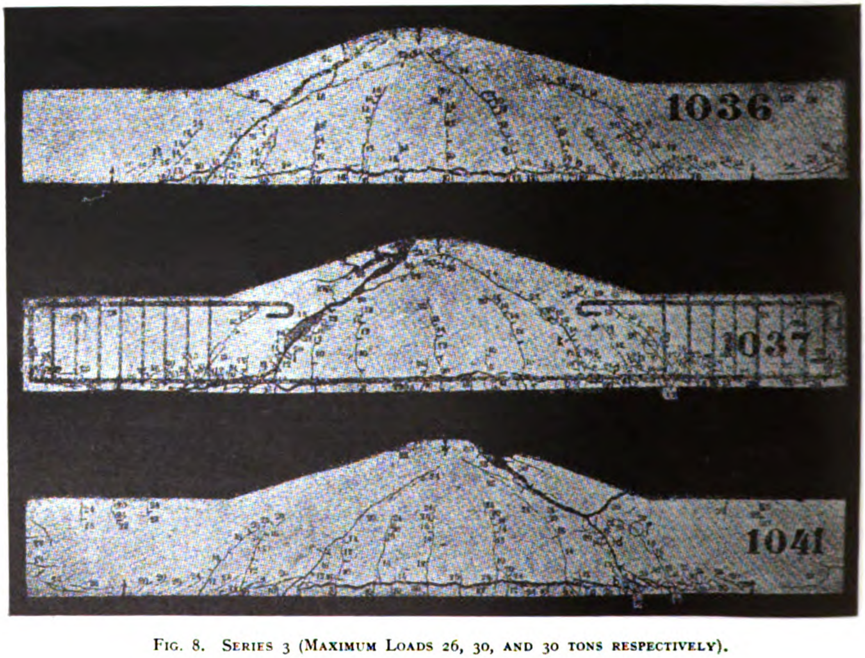


Fig. 8. Series 3 (Maximum Loads 26, 30, and 30 tons respectively).

SHEAR TESTS IN ARCHED BEAMS. (*See p.*599.)

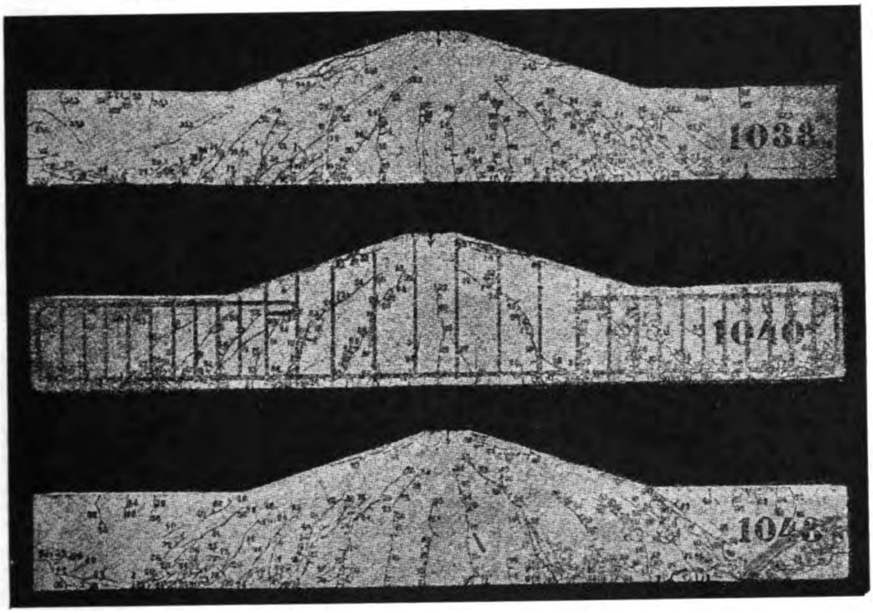


FIG. 9. SERIES 4 (MAXIMUM LOADS 35, 38, AND 40 TONS RESPECTIVELY).

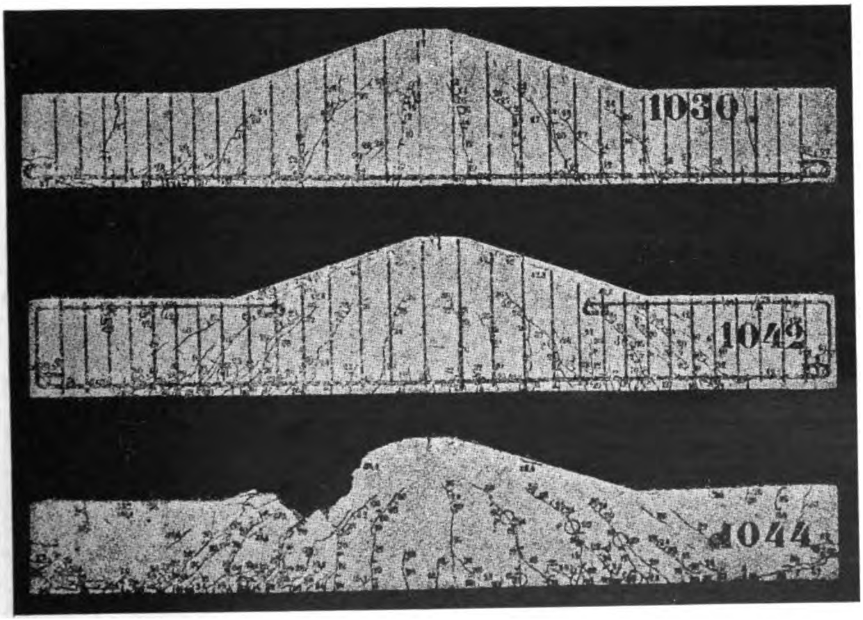


FIG. 10. SERIES 5 (MAXIMUM LOADS 28, 43, AND 38 TONS RESPECTIVELY).
[Note.—Beam No. 1030 does not contain the *a* rods.]

SHEAR TESTS IN ARCHED BEAMS. (*See p.* 599.)

PROPOSED STANDARD SPECIFICATIONS FOR CONCRETE FLOORS.

THE following abstracts are from the "Standard Specifications for Concrete Floors," prepared by the Committee of the American Concrete Institute on Concrete Floor Finish. The specifications apply to floors in buildings, whether subjected to moderate or heavy traffic, and cover the laying and finishing of the floor; also its protection during early hardening.

For those desiring to embody the specifications in their general specifications covering a particular piece of work, the following outline of the paragraphs necessary to meet different conditions is given :—

FLOORS LAID ON GROUND :—MODERATE OR LIGHT TRAFFIC.

Two-Course—Paragraphs 1–19 (except 8); 34–52; 54–57.

One-Course—Paragraphs 1–19 (except 8); 34–47; 58–62.

FLOORS LAID ON GROUND :—HEAVY TRAFFIC.—

Two-Course—Paragraphs 1–19; 34–51; 53–57.

REINFORCED CONCRETE FLOORS :—MODERATE OR LIGHT TRAFFIC.—

Paragraphs 1–26 (except 8); 28–33.

REINFORCED-CONCRETE FLOORS :—HEAVY TRAFFIC.—

Paragraphs 1–25; 27–33.

MATERIALS.

1.—The cement shall meet the requirements of the current Standard Specification.

2.—In no case shall aggregate containing frost or lumps of frozen material be used. Crusher-run stone, bank-run gravel, or mixtures of fine and coarse aggregate prepared before delivery on the work shall not be used.

3.—Fine aggregate shall consist of natural sand or screenings from hard, tough crushed rock or gravel consisting of quartz grains or other hard material, clean and free from any surface film or coating and graded from fine to coarse, with the coarse particles predominating.

4.—Fine aggregate, when dry, shall pass a screen having 4 meshes to the linear in.; not more than 25 per cent. shall pass a sieve having 50 meshes per linear in. and not more than 5 per cent. shall pass a sieve having 100 meshes per linear in.

5.—Fine aggregate shall not contain injurious vegetable or other organic matter as determined by the colorimetric test nor more than 5 per cent. by volume of clay or loam. If there is more than 7 per cent. of clay or loam by volume in 1 hour's settlement after shaking in 100 per cent. excess of water, the material represented by the sample shall be rejected.

6.—Fine aggregate shall be of such quality that mortar composed of 1 part Portland cement and 3 parts fine aggregate, by weight, when made into briquets, shall show a tensile strength at 7 and 28 days at least equal to the strength of briquets composed of 1 part of the same cement and 3 parts sand, by weight. The percentage of water used in making the briquets of cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the sand briquets of standard consistency.

7.—Coarse aggregate shall consist of clean, hard, tough, crushed rock or pebbles graded in size, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall range from 1½ in. down, not more than 5 per cent. passing a screen having 4 meshes per linear in. and no intermediate sizes shall be removed.

8.—No. 1 aggregate for the wearing course shall consist of clean, hard, tough, crushed rock or pebbles, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. It shall pass when dry a screen having ⅜ in. openings and not more than 10 per cent. shall pass a screen having 4 meshes per linear in.

9.—Water shall be clean, free from oil, acid alkali or vegetable matter.

10.—Colouring materials shall be such as will not impair the strength of the concrete.

11.—The reinforcing metal shall meet the requirements of the current standard specifications for steel reinforcement. It shall be free from excessive rust, scale, paint or coatings of any character which will tend to reduce or destroy the bond.

12.—The joint filler shall be a suitable compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather; or prepared strips of fibre matrix and bitumen. The strips shall be ½ in. in thickness and their width shall at least equal the full thickness of the slab.

CONSTRUCTION.

13.—The method of measuring the materials for the concrete or mortar, including water, shall be one which will ensure separate and uniform proportions of each of the materials at all times.

14.—All concrete shall be mixed by machine, except under special conditions. The ingredients shall be mixed to the specified consistency, and the mixing shall continue for at least 1 minute after all the materials are in the drum. Raw materials shall not be permitted to enter the drum until all the material of the preceding batch has been discharged.

15.—When it is necessary to mix by hand, the materials shall be mixed dry on a watertight platform until the mixture is of uniform colour, the required amount of water added, and the mixing continued until the mass is of uniform consistency and homogeneous.

16.—Retempering of mortar or concrete which has partially hardened (that is, mixing with or without additional materials or water) shall not be permitted.

17.—As soon as the finished floor has hardened sufficiently to prevent damage thereby, the floor shall be covered with at least 1 in. of wet sand, or 2 in. of sawdust, which shall be kept wet by sprinkling with water for at least 10 days.

18.—The freshly-finished floor shall be protected from hot sun and drying winds until it can be sprinkled and covered as above specified. The concrete surface must not be damaged or pitted by raindrops, and the contractor shall provide and use when necessary sufficient tarpaulins to completely cover all sections that have been placed within the preceding 12 hours.

19.—If at any time during the progress of the work the temperature is, or in the opinion of the architect or engineer will, within 24 hours, drop to 40 deg. Fahr., the water and aggregates shall be heated and precautions taken to protect the work from freezing for at leas 5 days.

REINFORCED-CONCRETE FLOORS.

For reinforced-concrete floors the following will apply in addition to paragraphs 1 to 19 inclusive.

20.—*Forms.*—The forms shall be substantial, unyielding, and so constructed that the concrete will conform to the designed dimensions and contours, and shall also be tight to prevent the leakage of mortar. The supports for floors shall not be removed without the consent of the engineer in charge. Permanent shores shall be placed in such a manner as to assure safety of the floors after temporary supports are removed.

21.—*Reinforcement.*—Reinforcing metal shall be mechanically held in position so that it will not become disarranged during the depositing of the concrete. Whenever it is necessary to splice tension reinforcement, the character of the splice shall be such as will develop its full strength. Splices at points of maximum stress shall be avoided. Splicing by lapping bars without contact and with space between bars along the overlap equal to twice the thickness of the bars is preferable to mechanical splices or clamps.

22.—*Concrete Slab.*—The concrete shall be mixed in the proportions by volume of 1 cu. ft. Portland cement, 2 cu. ft. of fine aggregate, and 4 cu. ft. of coarse aggregate.

23.—Only sufficient water shall be used to produce a workable plastic mix, which will flow sluggishly into the forms and around the reinforcement and which can be conveyed from the mixer to the forms without the separation of the coarse aggregate from the mortar.

24.—The concrete shall be placed in a manner to ensure a smooth ceiling, and thoroughly worked around the reinforcement and into the recesses of the forms. Concrete shall be deposited in its final position as soon as possible after mixing. It shall be struck off to a surface at least 1 in. below the established grade of the finished surface of the floor. Workmen shall not be permitted to walk on freshly-laid concrete, and if sand or dust collects on the base it shall be carefully removed before the wearing course is applied.

25.—When it is necessary to make a joint in a floor slab, its location shall be designated by the architect or engineer; joints to be vertical.

WEARING COURSE.

26.—(Mixture No. 1.)—The mortar shall be mixed in the proportions of 1 cu. ft. of Portland cement and 2 cu. ft. of fine aggregate. The minimum thickness shall be ¾ in.

27.—(Mixture No. 2.)—The mortar shall be mixed in the proportions of 1 cu. ft. of Portland cement, 1 cu. ft. of fine aggregate, and 1 cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be ¾ in.

28.—The mortar shall be of the driest consistency possible to work with a sawing motion of the strikeboard.

29.—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard.

Note.—When placing the wearing course after the concrete slab has hardened, eliminate paragraph 29 and substitute paragraphs 30 and 31.

30.—The surface of the slab shall be thoroughly roughened by picking or other means and cleaned of all dirt and débris.

31.—The slab shall be thoroughly moist but free from pools of water when the grout and mortar for wearing course is placed. A neat cement grout shall be brushed on the surface of the slab, the wearing course immediately applied and brought to the established grade with a strikeboard. Grout and mortar shall be used within 45 minutes after mixing with water.

32.—After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel-troweled, but excessive working shall be avoided. A mixture of dry cement, sand and No. 1 aggregate may be applied to the fresh concrete of the base for a wearing course, but in no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface of the wearing course to absorb moisture or to hasten the hardening.

33.—If artificial colouring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform colour. In no case shall the amount of colouring exceed 5 per cent. of the weight of the cement.

PLAIN CONCRETE FLOORS.

For plain concrete floors the following will apply in addition to paragraphs 1 to 29 inclusive.

34.—*Subgrade.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding 6 in. in thickness. The subgrade shall be thoroughly tamped until it is brought to a firm, unyielding surface.

35.—All fills shall be made in a manner satisfactory to the architect or engineer. The use of muck, quicksand, soft clay, spongy or perishable material is prohibited.

36.—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer.

37.—The subgrade shall not be less than —— in. below the finished surface of the floor.

Note.—Subgrade is to be —— in. below the finished surface of the floor when subbase is not required, and at least —— in. below when subbase is required.

38.—*Subbase.*—Only clean, hard material, such as coarse gravel or cinders free from ash or particles of unburned coal, shall be used in the subbase.

39.—The material shall be spread on the subgrade, thoroughly rolled or tamped to a surface at least —— in. below the finished grade of the floor. On fills, the subbase shall extend the full width of the fill.

40.—While compacting the subbase, the material shall be kept thoroughly wet, and shall be in that condition when the concrete is deposited.

Note.—Omit these sections when subbase is not required.

41.—*Forms.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape.

42.—The forms shall be well staked or otherwise held to the established lines and grades, and their upper edges shall conform to the established grade of the floor.

43.—All wood forms shall be thoroughly wetted and metal forms oiled or coated with soft soap or whitewash before depositing any material against them. All mortar and dirt shall be removed from the forms that have been previously used.

44.—*Limiting Conditions.*—The slabs or independently-divided blocks when not reinforced shall have an area of not more than 100 sq. ft., and shall not have dimensions greater than 10 ft. Larger slabs shall be reinforced as hereinafter provided.

45.—The thickness of the floor shall be not less than —— in.

46.—When required by the architect or engineer in charge, a ½ in. space or joint shall be left between the floor and the walls and columns of the building, to be filled with the material specified under "Joint Filler."

47.—Where required, the edges of the slabs at the joints shall be protected by metal. Unless protected by metal, the upper edges of the slabs shall be rounded to a radius of ½ in.

TWO-COURSE PLAIN CONCRETE FLOOR.

48.—*Concrete Base.*—The concrete shall be mixed in the proportions by volume of 1 cu. ft. of Portland cement, 2½ cu. ft. of fine aggregate, and 5 cu. ft. of coarse aggregate.

49.—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

50.—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections of the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established elevation of the floor. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. If dirt, sand, or dust collects on the base it shall be removed before the wearing course is applied. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. In no case shall concrete be deposited upon a frozen subgrade or subbase.

51.—Slabs having an area of more than 100 sq. ft. shall be reinforced with wire fabric, or with plain or deformed bars. The reinforcement shall have a weight of not less than 28 lb. per 100 sq. ft. The reinforcement shall be placed upon and slightly pressed in the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

52.—*Wearing Course.*—The wearing course shall be mixed in the proportions of 1 cu. ft. of Portland cement to 2 cu. ft. of fine aggregate. The minimum thickness shall be ¾ in. Or

53.—The wearing course shall be mixed in the proportions of 1 cu. ft. of Portland cement, 1 cu. ft. of fine aggregate, and 1 cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be ¾ in.

54.—The mortar shall be of the driest consistency possible to work with a sawing motion of the strikeboard.

55.—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard. In no case shall more than 45 minutes elapse between the time the concrete for the base is mixed and the wearing course is placed.

56 and 57.—As paragraphs 32 and 33.

ONE-COURSE FLOOR.

58.—The concrete shall be mixed in the proportions of 1 cu. ft. of Portland cement to not more than 2 cu. ft. of fine aggregate and not more than 3 cu. ft. of coarse aggregate, and in no case shall the volume of the fine aggregate be less than ½ the volume of the coarse aggregate. A cu. yd. of concrete in place shall contain not less than 6·8 cu. ft. of cement.

59.—The materials shall be mixed with sufficient water to produce a concrete which will hold its shape when struck off with a strikeboard. The consistency shall not be such as to cause a separation of the mortar from the coarse aggregate in handling.

60 and 61.—As paragraphs 50 and 51.

62.—After the concrete has been brought to the established grade by means of a strikeboard, and has hardened somewhat, but is still workable, it shall be floated with a wood float in a manner which will thoroughly compact it and provide an even surface. When required, the surface shall be steel troweled, but excessive working shall be avoided. Unless protected by metal, the surface edges of all slabs shall be rounded ½ in.

BOOK REVIEWS.

"Foundations, Abutments, and Footings."—By Hool and Kinne.

London: McGraw-Hill Publishing Co., Ltd. Price 20s. net.

THIS is the first of a series of structural books to be completed in six volumes. The two names above given are those of the chief editors, and the books themselves are compiled by a staff of about twenty associate editors—we should call them authors—which include some very eminent names.

The preparation of works of this kind by a large number of people in small sections has obvious advantages and certain disadvantages. Among the former are, of course, those of having a specialist on his own subject and of making the volumes cover a wide field. Among the latter are the inevitable overlapping and omissions, and the loss of continuity of thought and treatment. Examples of overlapping appear to evidence themselves even on a cursory examination — the second volume, "Structural Members and Connections," contains 140 pages on reinforced concrete, while the fifth volume deals entirely with "Reinforced Concrete and Masonry Structures."

We are told that "each volume is a unit in itself, as references are not made from one volume to another by section and article numbers. This arrangement allows the use of any one of the volumes as a text in schools and colleges without the use of any of the other volumes." It is difficult to see how Vol. V can fail, under this arrangement, to include much of the material already included in Vol. II, and this difficulty must occur in other places if this ideal is to be maintained.

The list of authors (or "associate editors") is a guarantee that the work will be interesting and of a high standard, and we hope the following notes will not be taken anywhere as intended to suggest otherwise. Clearly on such subjects we are all entitled to our opinions.

In the first volume the subdivisions divide the work up into Soil Investigation, Excavation, Foundations, Spread Footings, Underpinning, Special Foundations, Bridge Piers and Abutments, Legal Provisions. The work is typically American, and many of the descriptions of methods of excavation are more common in the States than elsewhere. The descriptions are generally clear and well illustrated.

On page 106 (*et sequitur*) we have Meem's theory of earth pressure, which gives a curve of pressure on a vertical face rising to a maximum about half-way down and then dropping to zero at the bottom. This is based on ingenious but, we think, misleading reasoning, and might lead to very serious results. According to the theory the triangular wedge of earth behind the front face and the line drawn from the base at the angle of repose is divided by a second line, O,B, and only the earth up to this line is considered as exerting pressure, the rest being supported by arching action. The author says: "The actual determination of just where and what this line is has not been made, nor can it be except by experiments on a large scale." We should have thought that under these circumstances it would be obvious that the theory should be one for consideration and experiment, but not as a basis for calculations involving the safety of human life. The argument that the bottom plank of an excavation may be removed without causing failure is no argument that the pressure here is zero, because so may the middle plank halfway up, and yet this is where the author claims the pressure is a maximum.

Nor do we like the author's treatment of pressure of water-bearing soil, being a combination of the previous diagram with a diagram of the Rankine shape. In this the authors seems to miss the point—the all-important point—that with water the angle of repose of the soil will generally change radically owing to the lubricating and softening effect of water, not to mention the pressure produced by swelling clay as it absorbs water. Altogether we consider this treatment misleading.

We are surprised to find (p. 141) the driving of test piles recommended without the addition of a note drawing attention to the unreliability of all pile formulæ—as is sufficiently exemplified by the conflicting results, varying by many hundreds per cent. of different formulæ—unless based on actual loading experiments on piles in similar strata and other circumstances.

Nor is the remark (p. 141) " wood piles cannot be designed but must be taken as they grow " very useful. A 12 in. by 12 in. Oregon pile can be designed just as much as a 12 in. by 12 in. concrete pile, or as much as a bridge timber. We need not take it as it grows, for we can cut it to size and make our own selection.

On p. 221 we have Talbot's formula for the moment on a footing. This formula gives a smaller moment than that determined by static analysis, ignoring friction on the soil. It was derived from the results of laboratory experiments in which friction between the base and the soil was considerable. In practice, particularly on clay, this may be unreliable and we think it would be wise not to depend on this too much.

On p. 253, *Fig.* 1 is described as a case of a building standing " apparently against the laws of nature." This is only because our " laws of nature " are so bad. In the case in point a small tensile strength in the brickwork is all that is required to make the structure comply. Arching action is given as the explanation, but arching action without tensile strength in the brickwork would not have made the structure stable as the thrusts from the two walls would have had a large unbalanced outward resultant.—O. F.

"Structural Members and Connections."—By Hool and Kinne.
London : McGraw-Hill Publishing Co., Ltd. Price 30s. net.

This is the second volume of the series commented on above, and the general notes on the whole series apply to it. The following matters of detail strike one on perusing it.

On p. 299 we are told " cast-iron columns are suitable only for small buildings of non-fireproof construction." As structural steel is known to be useless in fire unless protected by concrete we do not see why cast iron is any worse. If protected, it should be just as fireproof.

On p. 301 is given a series of formulæ, but it is not sufficiently emphasised that the last one is of ultimate stresses and the remainder of working stresses. This makes a difference.

On p. 305 we find steel columns—evidently these are intended to mean stanchions, and we fear the authors have not had the advantage of Mr. Etchells' teaching that a " column " is necessarily something round.

On p. 335 diagrams of cleats are given ; it would have added to their usefulness if the safe loads were given.

On p. 362 the strength of timbers is worked out for a stress of 1,500 lb. per sq. in., and on p. 370 onwards at a stress of 1,800 lb. per sq. in. These are very high stresses, and on referring to the only reference made to the strength of timber (Appendix D, p. 597), we find the maximum working stress recommended as 1,300 for longleaf pine and much less for most other timbers.

The steel connections between timber members (p. 425) are interesting.

The last portion on reinforced concrete members, p. 431 to 572, is considerably lacking in balance. For example, all the usual complicated formulæ are given for determining stresses in beams with and without compression steel, when the moment is known, as if a small error to secure greater simplicity would be serious. Yet the only reference to determining the value of the moment seems to occur on p. 454, where $\frac{WL}{12}$ at midspan and at the support is stated to be almost universal—though it is added that the J.C. 1921 report recognises a positive moment of $\frac{WL}{16}$ for this case, which is more reasonable. Apparently, then, errors of 33 per cent. in the determination of the moments do not matter, and the fact that the centre moments may vary from $\frac{WL}{12}$ to $\frac{WL}{24}$ under various conditions is not worthy of mention, far less of study.

The same applies later where (pp. 541 to 567 inclusive) are devoted to curves governing the design of columns under combined direct stress and bending. Yet nowhere can one find any statement of what bending moment a column should be designed for; the nearest we get to it seems to be on p. 492: "Bending in the columns due to unbalanced floor loading must always be taken into account in design." This is not particularly helpful.

From the point of view of English readers it is, of course, a disadvantage that all the authorities and regulations quoted are American, and generally it would be more helpful if a little less space were taken up in systematic design sheets and diagrams and a little more devoted to the theoretical or experimental study with which all design should be in touch.—O. F.

EFFECT OF TRASS AND STONE DUST ON CONCRETE.

The effect of trass and other stone dust on cement and concrete was examined by means of a long series of tests, with the result that: (i) The volume-weight of the concrete containing trass, limestone dust, and other stone dust is increased, and the mixture is therefore denser than when they are absent. (ii) Finely-ground trass did not increase the strength of the product more than stone dust when the concrete was kept moist until fully cured. If it was allowed to become dry before curing was complete, the use of trass gave superior results to stone dust. In comparing the effect of different stone dusts, those were found to be best which were most absorbent and therefore remained moist longer than the others, and in this way enabled the curing of the concrete to proceed further without the external application of water.

These results are in opposition to those of Hamblock, Gary, and others, who found that trass combined with the free lime in the cement and formed insoluble calcium hydrosilicates, which could not be the case with some other stone dusts, such as ground limestone. Hence, trass must be superior to most other powders if it contains a sufficient proportion of silica in a form in which it can combine with the free lime.

In the production of concrete to resist the action of water or other fluids it is most important to obtain a mass of maximum density, as this will offer the minimum exposed surface. It is equally important that the concrete should be "cured" in the presence of sufficient water, though this must not be present in so large a quantity as to wash out any of the constituents. For this reason exposure to moist air is often preferable to immersion in water.—*Beton u. Eisen.*

Concrete Centre Lines on Road Curves.

A recent issue of *The Motor* contains an account of a novel method adopted for preventing accidents at a dangerous curve at Jockey Hill, Sutton, Birmingham. This consists of a strip of concrete, 3 in. wide, let into the road to indicate the centre, so that drivers can be fairly sure that, providing other vehicles observe the same precaution, there will be no danger of a head-on collision if they keep to the left of the guiding line. Such a strip is permanent, and shows up quite plainly even when the road is wet. "It seems to us," states our contemporary, "that when new surfaces are laid on curves and steep hills this question of inserting a strip will have to be seriously considered, for it is a comparatively simple method of emphasising that a great risk is involved in attempting to pass another car, and that the 'safety first' method will be for all drivers at such spots to keep to the left of the safety line."

A TASMANIAN MULTIPLE-ARCH DAM.

THE multiple-arch dam illustrated on pp. 610–11 has been constructed by the Hydro-Electric Department of Tasmania at the Great Lake, and forms the principal feature in connection with the conversion works of the Great Lake hydro-electric scheme. It is claimed to be the only dam of its type in the British Empire, and also to be the longest in the world, being 1,182 ft. from buttress to buttress. This type of dam was adopted after careful consideration of all possible alternatives, and designs and estimates were made for alternative spans of 30, 40 and 50 ft., the 40-ft. span being adopted as the most economical.

The dam has an arch height of 40 ft. and is intended to store water to 35 ft. above the sill of the sluice gates with 5 ft. freeboard. It consists of 27 arches of 40 ft. span each, approximately 100 ft. of abutments, and cut-off wall. The securing of foundations caused considerable trouble in two places, it being necessary to go to a depth of 50 ft. below ground level. Great precautions had to be taken in connection with these excavations in view of the fact that the Great Lake conserved by the old gravity dam was within a few feet of the site of the excavations.

The arches are inclined at an angle of 60 deg. between the horizontal and the extrados (water face) of the arch. The thickness of the arch is uniform in planes normal to the extrados, but increases uniformly at the rate of 0·019 ft. per ft. of depth measured vertically, the thickness of the crown at the top of the arch being 1 ft. The extrados is a true circle in the horizontal plane, with a radius of approximately 22 ft. The arches were designed in accordance with the elastic theory, and reinforcement was provided over the full span of the intrados face and over the portion of the extrados face at the springing in order to provide for temperature variation.

The buttresses are tapered and reinforced. They are further supported by two lines of beams 10 ft. 6 in. apart vertically, the tops of the buttresses being tied in by a pair of floor beams supporting the footway which runs the whole length of the dam.

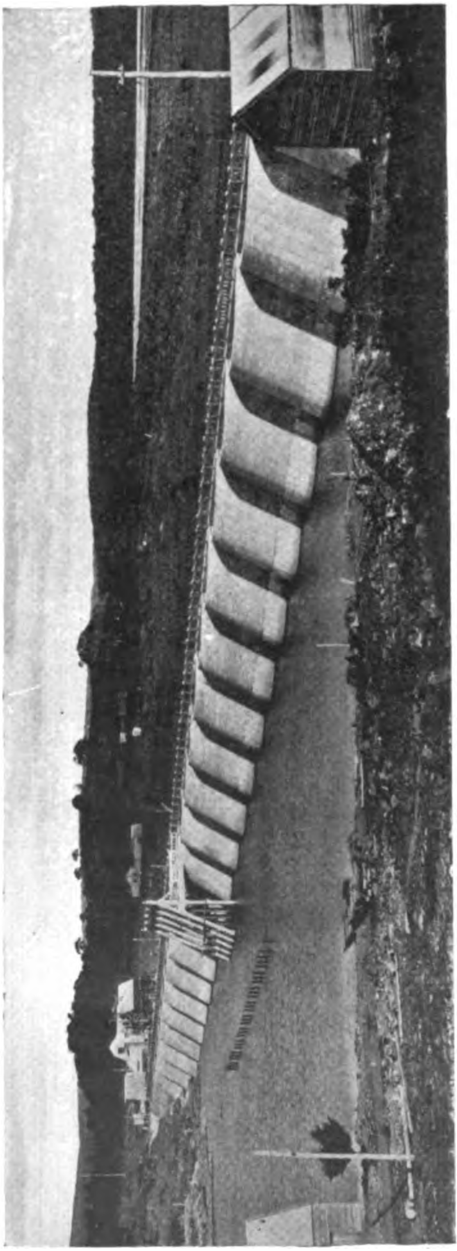
The outlet structure consists of a concrete bell-mouth divided into four separate compartments, each compartment tapering into a 4 ft. by 3 ft. cast-iron sluice. The discharge is conducted clear of the buttresses by concrete draining walls with a concrete bottom, and then by a short rock channel to the original water course.

The concrete for the dam was mixed and placed by a moving tower 80 ft. high, running the full length of the dam on a track 20 ft. centres. The tower contained bins with a capacity equal to one day's requirement of sand, stone, and cement. The working plant on the job was driven by electric power transmitted from Waddamana.

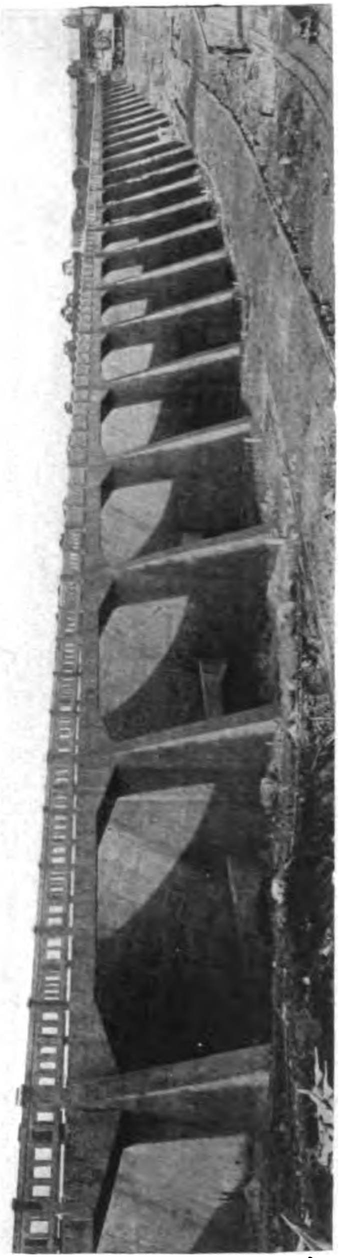
The dam was designed and constructed by the staff of the Hydro-Electric Department, under Mr. J. H. Butters, Chief Engineer and general manager; Mr. C. C. Halkyard was in direct charge of the design work, and Mr. A. H. Bastow was in direct charge of the construction, with Mr. W. E. Maclean as resident engineer. These particulars and the accompanying illustrations are given by courtesy of the *Commonwealth Engineer.*

Industrial Standardisation.

A SECOND unofficial conference of Secretaries of the various standardising organisations now in existence has recently been held in Zurich, Switzerland. It was called by the Secretary of the Swiss Committee, and the Secretaries of the Standards Associations or Committees in the following countries were present: Austria, Belgium, Canada, Czechoslovakia, France, Germany, Great Britain, Holland, Italy, Norway, Sweden, Switzerland, U.S.A. Mr. Zollinger, the Secretary of the Swiss Committee, was appointed the Chairman of the Conference, and Mr. le Maistre, the Secretary of the British Engineering Standards Association, was appointed Vice-Chairman. The Secretaries of a number of organisations which have come into being since 1921 were present and reports on the position in their countries with regard to standardisation were received with much interest. A general discussion took place in regard to the organisation and methods of working, the practical application of the standards, and also the extent to which international collaboration is taking place at the present time and the possibilities of making it more effective.

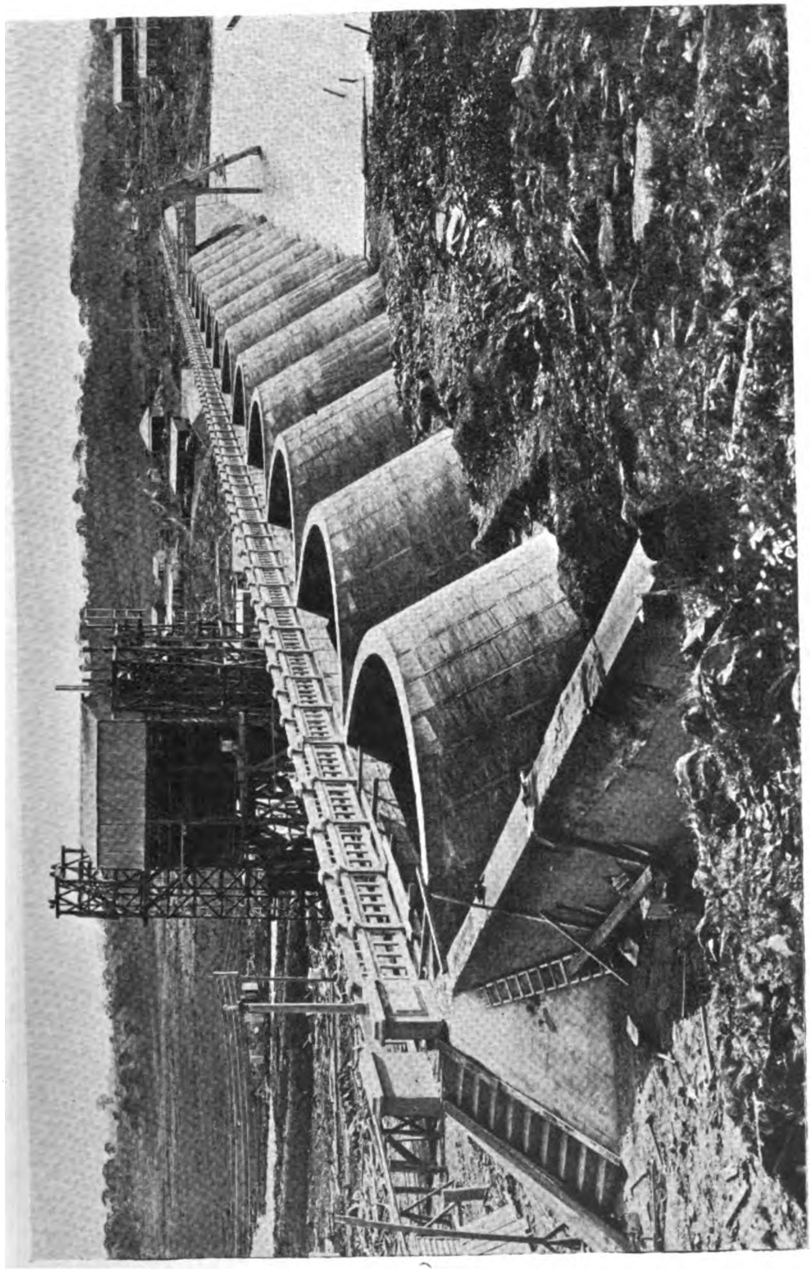


Extrados Face.



Intrados Pack.

MULTIPLE-ARCH DAM IN TASMANIA. (See p. [illegible])



MULTIPLE-ARCH DAM IN TASMANIA: EXTRADOS FACE. (*See p.* 609.)

PORTABLE CONVEYORS

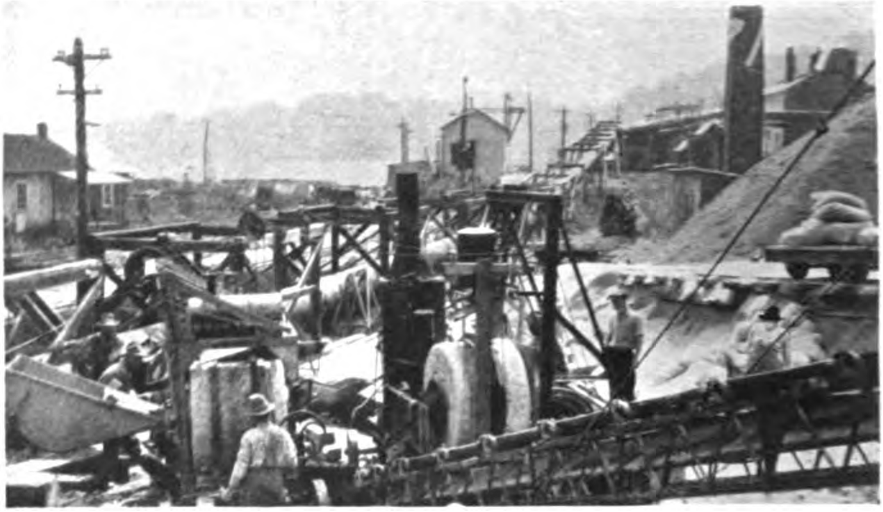


Fig. 1.

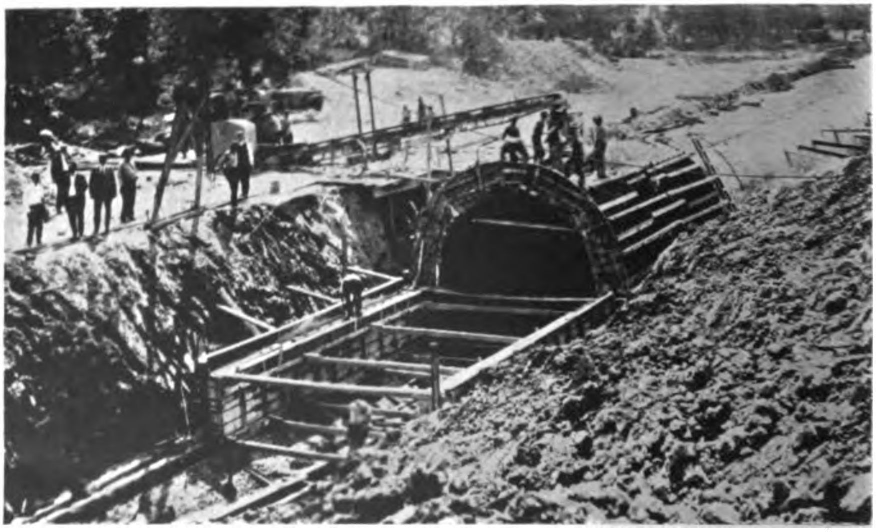


Fig. 3.

Recently several American construction engineers have been using portable belt conveyors for placing wet concrete with practical results. By replacing wheelbarrow gangs with the conveyors the number of men on some jobs has been reduced from as many as twenty-seven to only ten. On an Illinois sewer job forms containing 37 cu. yds. of concrete were placed in two hours with a conveyor. It was found that the belt would carry concrete of any consistency. The conveyor is equipped with an 18-in. rubber belt running on troughing carriers. At the discharge end a spout catches all the concrete and discharges it

NDLING WET CONCRETE.

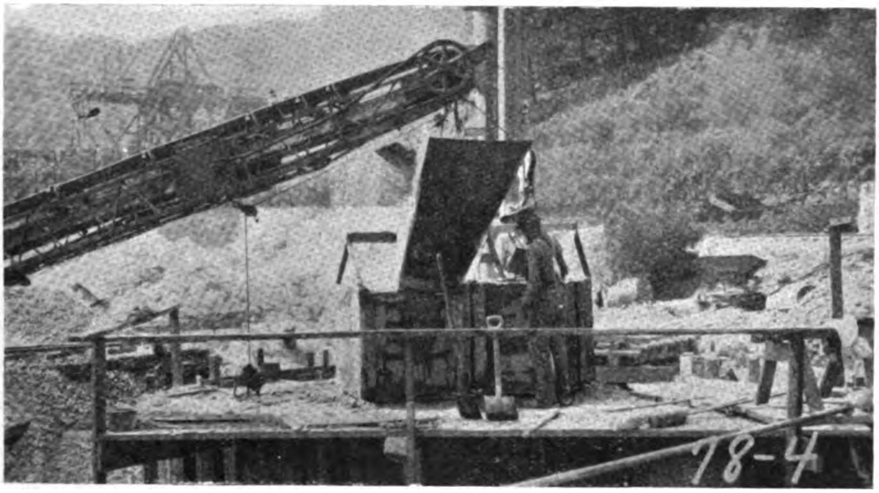


FIG. 2.

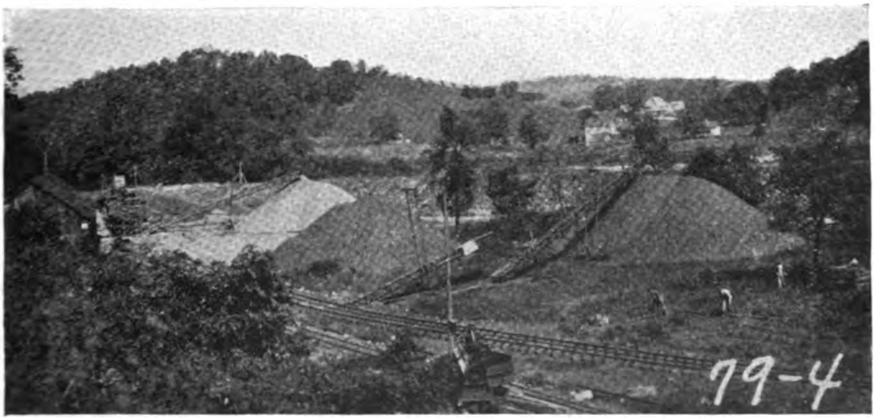


FIG. 4.

straight down so as to avoid segregation of materials in the mix. The concrete is discharged directly from the mixer into the hopper end of the conveyor. *Fig.* 1 shows the manner in which the mixing plant was placed for employing a belt conveyor for placing concrete for the base of a large chimney, and *Fig.* 2 shows the discharging of the concrete from the conveyor. *Fig.* 3 shows the use of portable belt conveyors on sewer construction. When the belt conveyors are not required for placing concrete they are used for stacking aggregate, as shown in Fig. 4.

CONCRETE ROADS.

THE second edition of "Concrete Roads" is now ready, and the large number of advance orders received in response to the announcement that it was in the Press is evidence that the necessity for improved methods of road construction to withstand the wear and tear of the fast and heavy traffic of the present day is fully realised and that there is a considerable demand for information on the all-concrete road.

The chapters include: Concrete Roads in England, Scotland, Wales and Ireland, with latest Official Reports as to their condition, Weight of Traffic, Maintenance Cost, etc.; Concrete Roads in Industrial Works; Concrete Roads in Australia, New Zealand, Canada and America, with latest reports; The International Road Congress at Seville, May, 1923—Extracts from Reports and the Resolutions; the Bates Tests of Concrete and other Roads; Mechanical Appliances used in the Construction of Concrete Roads; Concrete Kerbs and Channelling; A suggested form of Specification for the making of Concrete Roads; Pavements, Fence Posts, Telegraph and Lamp Posts, etc.; Useful Hints and Data for making Concrete and the Choice of Aggregates; Press and other Opinions on Concrete Roads; etc. In the compilation of the book we have had the assistance of engineers who have had a great deal of practical experience in laying concrete roads, and whose work is amongst the most successful in the country.

The book is well illustrated throughout, and the particulars of the methods of construction used by different Borough Surveyors should be particularly valuable to those about to lay new roads—the volume is, indeed, the only publication in which this information has been collected together in a convenient form. The book is now ready (price 5s. net), and may be obtained from the Trade Manager, Concrete Publications, Ltd., No. 4, Catherine Street, Aldwych, London, W.C.1, by filling in and posting the form on p. xxiv.

A correspondent in Buenos Aires writes:—

SIR,—As you have devoted some space lately to the subject of concrete roads, perhaps the enclosed photograph may interest you. It shows a reinforced concrete road constructed some ten years ago in one of the suburbs of Buenos Aires. Although this road is exposed to a very intense sun and has to carry a heavy vehicular and motor traffic, being the main artery of a summer resort, it has shown no signs of deterioration beyond the usual wear and tear, and singularly little temperature cracks.

REINFORCED CONCRETE ROAD IN BUENOS AIRES.

RECENT PATENTS RELATING TO CONCRETE[*]

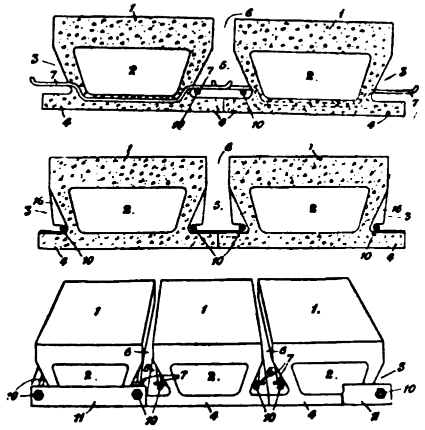
REINFORCED CONCRETE FLOORS.

[No. 195,485.—Pearson Bros. & Campbell, Ltd., and R. S. Pearson, 56, South Castle Street, Liverpool, April 5, 1923.]

This invention relates to floors, landings, steps, walls, and the like, of the type in which beams are formed of a plurality of blocks of cement concrete, clay, or the like, placed end to end, and held together longitudinally by tie rods, held by end nuts or the like, and placed within recesses in the sides or lower parts of the blocks.

According to the invention the blocks are placed in position in a row, and the tie rods in the recesses at the sides are held out of contact with the surfaces of the side recesses of the blocks by gauging or registering means; and when the beams with the tie rods are placed in their final position the spaces between the beams of blocks, and the spaces between the rods and the surfaces of the recesses in which they lie, are filled with grouting, so that it is ensured that practically the whole of the surfaces of the rods become completely embedded in the grouting.

The blocks (1) are provided with a hollow centre or core (2), and at each side with an angular recess (3), the lower portion of the block forming the bottom of the recess being in the form of a flange (4), which projects beyond the upper side face of the block, so that when the block beams are *in situ* the flanges (4) of the adjacent beams touch each other, and form the triangle space (5) beneath the neck (6).



No. 195,485.—Reinforced Concrete Floor.

The gauging device consists of wires or rods (7) of iron or steel cast in the blocks when making them in the mould, their ends being turned up when the grouting is filled into the angular spaces (5); they form also a transverse reinforcement.

The connecting and reinforcing rods are supported in position during the time a beam is being made up out of a number of the blocks, and subsequently thereto, and held in the required position, namely, a distance not less than $\frac{1}{8}$ in. from the surfaces of the sides and bottoms of the grooves or recesses (3).

In the construction the rods are held in position by means of projecting webs (16), formed on the sides of the blocks within the grooves or recesses (3) and the bottom flanges (4), and serving as gauges. The webs, as shown, lie above the rods (10), and where the rods lie in contact with the web supports they will be angle-shape so that the rods are only touched by a relatively sharp edge.

GROUTING MACHINE.

[No. 195502.—R. H. Annison, 16, Water Lane, London, E.C., April 5, 1923.]

This invention relates to grouting machines for forcing cement grout, of the type comprising a vessel or cylinder for the material which can be closed airtight, which is provided with an agitator, and which is in connection with a regulated supply of compressed fluid such as air, and has a regulated outlet pipe through which the material is forced.

According to the invention two or more cylinders are in separate connection with a mixing vessel from which the cylinders are filled, and in this way an apparatus is provided adapted to maintain a constant uniform flow.

In operation the material is placed in

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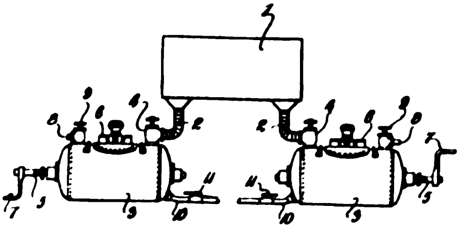
[*] These articles are prepared for this journal by a leading firm of Chartered Patent Agents; the dates given are the dates of acceptance.



No. 195,502. Grouting Machine.

the tank or mixer (1) and agitated with a suitable supply of water. One of the valves (4) leading to one of the cylinders (3) is then opened and the contents of the tank or mixer (1) passes to this particular cylinder (3). The valve (4) which was previously opened is now closed, and a fresh charge placed in the tank or mixer (1). Compressed air or the like is then admitted through the inlet (8) of the cylinder (3), which has been supplied with material by opening the valve (9) of this cylinder, and the charge in the cylinder is agitated by turning its handle (7), and discharged through the pipe (10) to the delivery nozzle and into the structure, soil, or the like, under treatment, the discharge being controlled by the particular valve (11).

The other cylinder (3) is then filled, whilst the first filled cylinder (3) is being discharged, and thus a continuous supply to the delivery nozzle, which might be common to both pipes (10), is assured. The mixing tank and the cylinders may be provided with heating or cooling coils or jackets.

CENTERING FOR FLOORS.

[No. 195859.—Moritz Kahn, 22, Cranley Gardens, S.W.7, April 12, 1923.]

This invention relates to centering for concrete floors of the type comprising inverted trough members of steel or other material and bearers which are provided with means for supporting the inverted trough members in position temporarily, to enable the floor or the like to be moulded *in situ*, which means permits the trough members to be withdrawn from beneath before the concrete has completely dried, leaving the bearers to support the concrete until it dries, when they in turn can be removed.

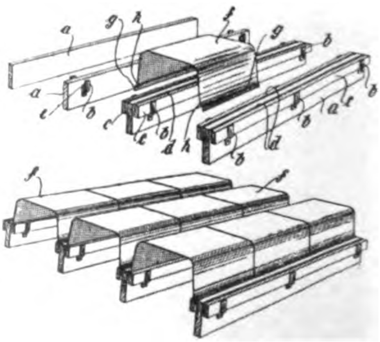
In erecting centering or falsework according to the invention, bearers (*a*) of wood or other suitable material are supported in any desirable manner from cross joists. A simple bearer (*a*) is shown in the illustration. Clips (*b*) are attached by bolts or the like to the bearers at suitable distances apart, and these clips are adapted to support runners (*c*). Suitable packing pieces are disposed between the bearers (*a*) and the runners (*c*) so as to leave gaps (*d*). These packing pieces may be constituted by parts (*e*) of the clips (*b*), or in some cases the lower part of the runners may be stepped or provided with a strip attached thereto for the purpose. Sections of inverted troughs (*f*) are preferably made of steel and having ribs (*g*) at the ends of the depending portions; these ribs are so spaced apart that they are adapted to come into adjacent gaps (*d*) of two adjacent bearers (*a*), which latter are suitably positioned for the purpose; (*h*) are somewhat curved flanges, which when a trough section is in position come on a level with the upper surface of the bearers (*a*).

When centering or falsework is completely erected, it presents the appearance shown in the lower illustration.

To enable spaces of different lengths to be readily dealt with, with troughs of uniform length, the troughs are arranged to telescope one into the other, the end of one trough being inserted into the slightly enlarged end of an adjacent trough.

A few days after pouring the concrete the clips with their bolts or other fastening means and the runners can be removed. This leaves the trough sections (*f*) free to be withdrawn from below, passing between the bearers (*a*).

The comparatively expensive troughs are now free to be used in another position, such as the next floor of a building, with other bearers (*a*).



No. 195,859.—Centering for Floors.

CONCRETE MIXER.

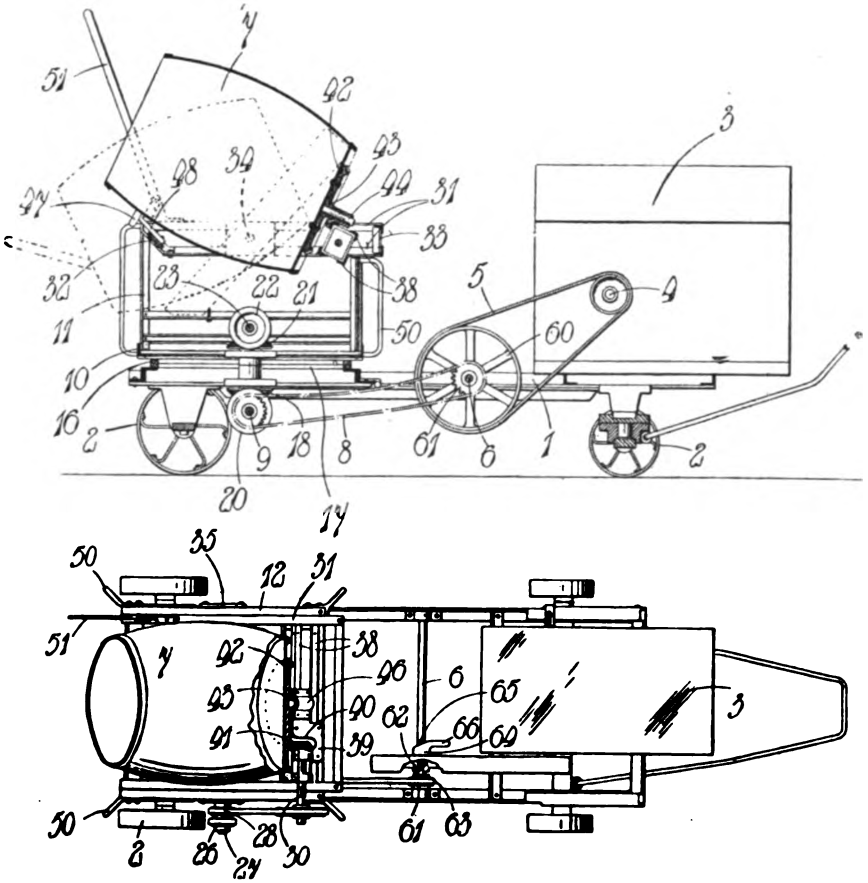
[No. 195881.—Parker, Winder and Achurch, Ltd., and G. P. Achurch, Broad Street, Birmingham, April 12, 1923.]

This invention comprises an improved concrete mixer of the type in which a barrel or the like, provided with internal blades, is rotated by power about its own axis to effect the mixing.

A power unit (3) mounted on the frame drives through belt and chain gearing a shaft (9), whence the drive is transmitted by bevel gearing (18) through a central vertical shaft and thence to a transverse shaft mounted in the frame (11) which is carried by the turntable (10) and is movable about a vertical axis around a circular track (16). One end of the shaft (23) projects beyond its bearing (24) at one side of the turntable, and is geared by means of the chain (25) to a chain wheel (26) mounted on a short fixed spindle (27) projecting outwardly from the side frame of the turntable. The chain wheel (26) is formed in one with a second chain wheel (28), and the drive is thus transmitted through the chain to the actuating shaft (30) which passes partially across the rear end of the mixing barrel (7).

In order that the barrel (7) may be tilted when required it is mounted in a pivoted cradle (31). The side members (31) have secured adjacent to their mid-points the pivot members (34) adapted to seat bearings (35) provided on the upper horizontal members (12) of the side frames of the turntable. The pivots (34) are provided with an enlarged portion (36) and attachment plate (37).

Towards its rear end the cradle (31) is provided with four transverse angle section members (38) arranged in box



No. 195,881.—Concrete Mixer.

form, and this structure is disposed parallel to the base of the barrel (7). Arranged above the members (38) is the actuating shaft (30) for rotating the barrel, and the inner end of this shaft is supported in a bearing block (39) which is secured to the members (38).

Towards its forward end the barrel (7) is freely supported on a pair of rollers (47) pivotally mounted on the brackets (48) secured to the inclined front end (32) of the cradle. It will be seen that the barrel (7) is capable of being rotated about its axial pivot (44) independent of the inclination of the cradle, since the cradle is pivotally mounted about the centre line of the chain wheels (26 and 28), which are fixed relative to the turntable and drive on to a spindle (30) mounted on the cradle.

CONCRETE PILES.

[No. 197430.—A. Hiley and E. Le Bas, Billiter Street, London. May 17, 1923.]

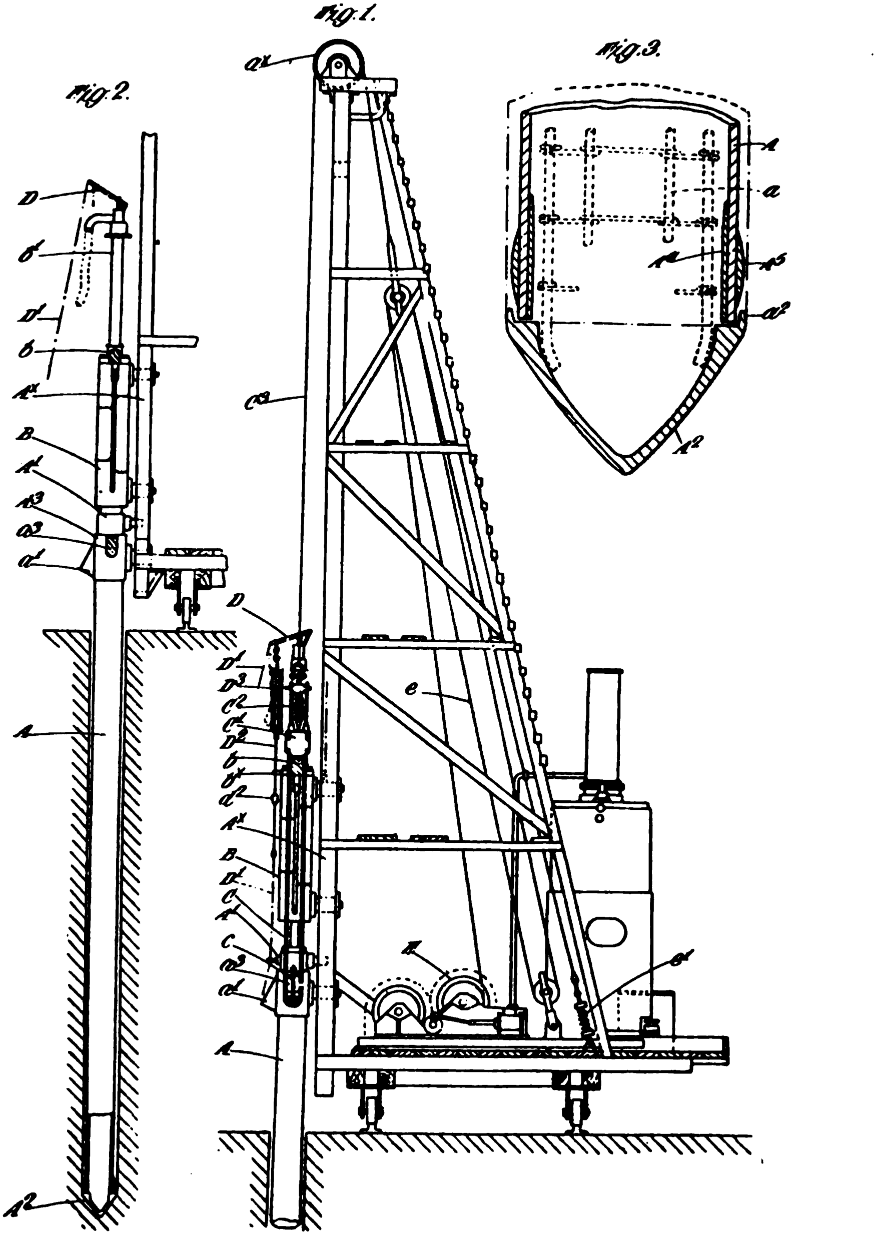
This invention relates to the construction of concrete piles or columns *in situ*, and has for its chief object to improve the method and apparatus forming the subject of Patent No. 179689.

In the improved method, according to the present invention, the concrete is both deposited from a preparatory tube and tamped and consolidated within the hole by vertical vibratory movements given to the tube by an upwardly and downwardly striking hammer. The pile is built up by a succession of operations each of which involves the depositing of a portion of the concrete from the tube by an upward movement of the latter occasioned by the upward blow of the hammer (which frees the concrete so that it falls down the tube by gravity) and the tamping of the deposited concrete by a downward movement of the tube under the combined influence of gravity and the force exerted by the downward blow of the hammer.

The tube (A) is operated upon by a hammer which, in the example shown, is one in which steam fluid is admitted to the heavy vertical cylinder (B) and by acting on the upper surface of a stationary piston causes the cylinder to perform its upward stroke, on the completion of which the steam is exhausted and the cylinder falls under the action of gravity and strikes its blow on the helmet or cap (A^1) of the tube (A). When the tube has been driven in this manner until a safe bearing resistance below the shoe (A^2) has been reached, the next stage of making the pile is commenced. The hammer and the driving cap (A^1) [which fits into and covers the head of the casting (A^3) at the top of the tube] are hoisted up the girders of the frame (A^4) to enable a cage (*a*) of steel reinforcement bars to be introduced down the tube, and a large charge of plastic concrete mixture is introduced into the tube through an aperture (a^1) in the casting (A^3), while its head is conveniently near the ground.

The hammer and cap are then lowered into their places on the tube and a pair of strong steel links (C, C) are fitted on to the tube casting (A^3), which is provided with suitable projections (a^3, a^3) to engage with and take the pull from the links. The upper parts of the links carry blocks (C^1, C^1) which serve the function of an anvil to receive the upward blows imparted by the cylinder head (*b*). The stroke for the purpose of extracting the tube is made approximately only one-fourth of the length of the stroke used in the hammer for the purpose of driving. Consequently a convenient space remains at the upper ends of the connecting links to allow of the fitting of relief springs (C^2, C^2) connected to the suspension ropes (C^3, C^3). The links (C, C) are provided with tie bolts (*c*, *c*) at their lower ends to bind them firmly to the projections (a^3, a^3). The links at their upper parts fit along the opposite sides of the cylinder head (*b*) and extend some distance clear above it, and at these free ends cross connections (c^1, c^1) are provided to retain the links correctly in their working positions. The links thus function as transmitters of upward impulses to the tube, and may be regarded as a portable skeleton framework or cover within which the kinetic member of the hammer travels to and fro in its successive strokes.

To press out smoothly and clear away any soil in the hole so that the concrete shall be able to emerge from the mouth of the tube into a near cavity, and preserve its good quality free from admixture of earth until it contacts with the walls of the hole, a smooth steel band (A^5) having a convex outer surface is fitted on the exterior of the tube near its lower extremity. This band, which is of substantially the same external diameter as the rim of the shoe, may be welded on to the tube and acts in the manner of a smoothing iron to finish off neatly the surface of the hole.



No. 197,430.—Concrete Piles.

The pile shoe (A^2) is formed with a rim (a^2) on its outer edge to allow the tube to fit therein, and has no upwardly projecting rim at its inner edge, so that it affords an uninterrupted passage for the concrete to flow outwards above the detachable shoe immediately any upward movement of the tube takes place under the blows of the hammer.

VIBRATING CONCRETE.

[No. 197735.—F. G. and E. L. Lynde, The Eaves, Belvedere, Kent. May 17, 1923.]

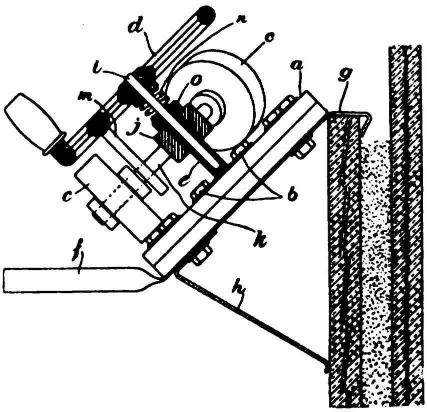
This invention relates to apparatus for producing vibrations in concrete while in a plastic state, so as to consolidate it prior to setting, as in ferro-concrete structures.

The base may be of any desired shape and material, and the interrupted surface is constituted by a series of projections acting as contacts for the movable member composed of rollers mounted in connection with, and actuated by, a common hand-operating device.

The base (*a*) is constructed as a rectangular compound board, and for handling it or holding it in use it is furnished with a handle (*f*), whilst for securing it in the operative position it is provided, where required, with a clamp (*g*) for engaging the side or edge of the mould.

The projections (*b–b*) are formed by the hexagonal heads of a series of set-pins inserted in the board (*a*) and disposed concentrically with the centre-piece or support (*e*), which is secured to and at the middle of the board. The centre-piece or support (*e*) is made hollow to accommodate the spindle (*i*) of the hand-wheel (*d*), and it also carries a boss (*j*) which carries three arms (*k, k, k*), each of which in turn supports a roller (*c*). Thus the boss (*j*), arms (*k, k, k*), and rollers (*c, c, c*) constitute a unit. The hand-wheel (*d*) is provided with a depending rod or finger (*m*) for co-operating with an appropriate arm (*k*). A spring (*n*) is interposed between the underside of the boss of the hand-wheel (*d*) and the adjacent end of the centre-piece or support (*e*), an abutment thereat being conveniently formed by a nut (*o*).

When the hand-wheel (*d*) is revolved the rollers (*c, c, c*) are traversed round the ring of projections (*b–b*) and roll over each in turn, whereby, due to the resulting blows or impacts, vibrations are set up or produced which are transmitted as rapid taps to the concrete directly through the interposed strut (*h*).



No. 197,735.—VIBRATING CONCRETE.

REINFORCING BARS.

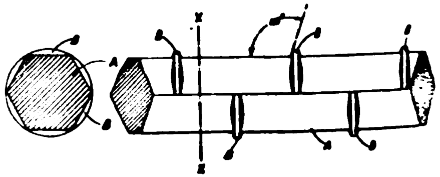
[No. 197768.—Robert Heath and Low Moor, Ltd., and W. E. Harvey, Biddulph Valley Iron Works, Stoke-on-Trent. May 23, 1923.]

The improved bar constructed in accordance with this invention has for its cross-section a polygon, bounded by straight lines and without re-entrant portions, and is provided at intervals along its length with ribs, a section through which normal to the bar has an external periphery circumscribing the polygon or a portion thereof; the ribs project beyond the polygon between the angles, but do not project at the angles.

The bars may be of hexagonal section, and the periphery of the normal section through the rib is a hole or part of the circle circumscribing or projecting beyond the hexagon; this enables a very small volume of metal to be taken in the ribs while ensuring sufficient projection for adhestion.

The bar (A) is provided at intervals with ribs (B) which are semicircular in form and arranged at convenient intervals along the length of the bar, the semicircle being preferably half of the circle circumscribing the hexagon.

The cross-section through the ribs by a plane intersecting the axis in the form shown, gives a figure bounded by three straight lines, the angle of the inclined sides with the horizontal being 105 deg. The bar (A) may have the well-known twisted form in which the generating lines of the hexagonal bar have a helical instead of a straight form.



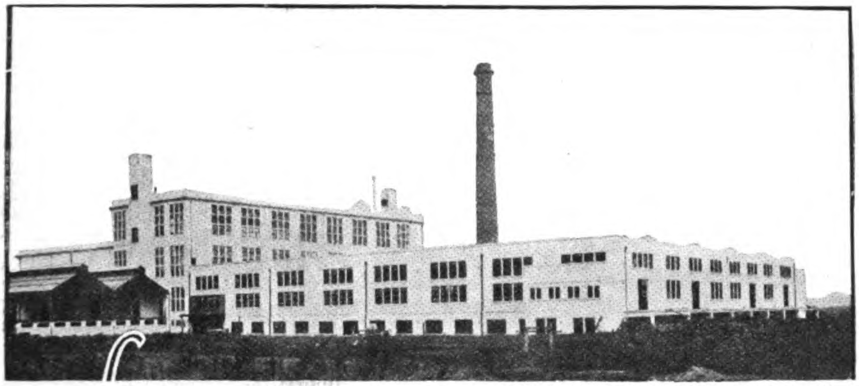
No. 197,768.—REINFORCING BARS.

REINFORCED CONCRETE WAREHOUSE AT MANCHESTER.

THE illustration below shows a new factory constructed entirely of reinforced concrete for Messrs. the Co-operative Wholesale Society, Ltd., to the design of their architect, Mr. F. E. L. Harris, A.R.I.B.A.

The site is at Irlam, nr. Manchester, and owing to the poor foundations a special raft construction was necessary, which was also carried out in reinforced concrete. The building was designed for very heavy loads and has tanks on the roofs. The total ground area covered is approximately 2 acres. The building is used as a soap works.

The reinforcement is on the "Lattice and Keedon" system of construction, provided by Messrs. Johnsons Reinforced Concrete Engineering Co., Ltd., of Manchester.



[*Mr. F. E. L. Harris, A.R.I.B.A., Architect.*

REINFORCED CONCRETE WAREHOUSE AT MANCHESTER.

"THE RISING CITY OF CONCRETE."

IN the course of an article describing the progress of the building work at the British Empire Exhibition at Wembley, *The Times* says: "If the life of London came to an absolute stop to-day, one wonders what the New Zealander who visited the deserted city five hundred years hence would think of present conditions as he would then set out to reconstruct them. . . . What would he make of Wembley and of the site of the British Empire Exhibition, which is to be opened next spring, if he saw it exactly as it stands at the moment? It is difficult to say. On one hand he would see, in its completed state, the greatest sports arena in the world. Gazing down from its terraces he would see the beginnings of stately buildings, cheek by jowl with wooden sheds, and vast stretches of steel and concrete intermingled with trees which have been spared from the axe. Possibly he might decide to dismiss the whole thing as an attempt to construct the eighth wonder of the world—after considering how the modern City of Concrete might have compared with the hanging gardens of Babylon. . . . This vast city of concrete is taking shape in a way that must every day make its designers proud of the great imaginative piece of work to which they have set their hands. . . . The actual buildings, of concrete and steel, like the rest of the exhibition, are practically complete. The Palace of Engineering, with its gross area of just under half a million square feet and its net area for exhibits of 325,000 square ft., is probably the largest concrete building in the world."

A NEW CONCRETE BARGE.

Concrete barges appear to be quite satisfactory on the Rhine-Rhone canal. They are owned separately by their skippers, and are usually towed by a horse.

Figs. 1 and 2 show a concrete barge built on the Zublin-Koller system by a firm which has built a considerable number of other concrete vessels.

The walls of the vessel are only 1 in. thick, and the reinforcement has to be placed very accurately so that it may be uniformly covered. The concrete is made in accordance with the result of a long series of tests, and has a specific gravity of 2·5 and a crushing strength of 14,200 lb. per sq. in. It is placed by means of movable shuttering, and special methods of work—the secrets of the builders—are used. The vessel was constructed without any external coating being necessary, the ordinary surface of the concrete being sufficiently hard and resistant. A highly porous aggregate is regarded as undesirable because of its low adhesion and the lesser strength of the resultant concrete.

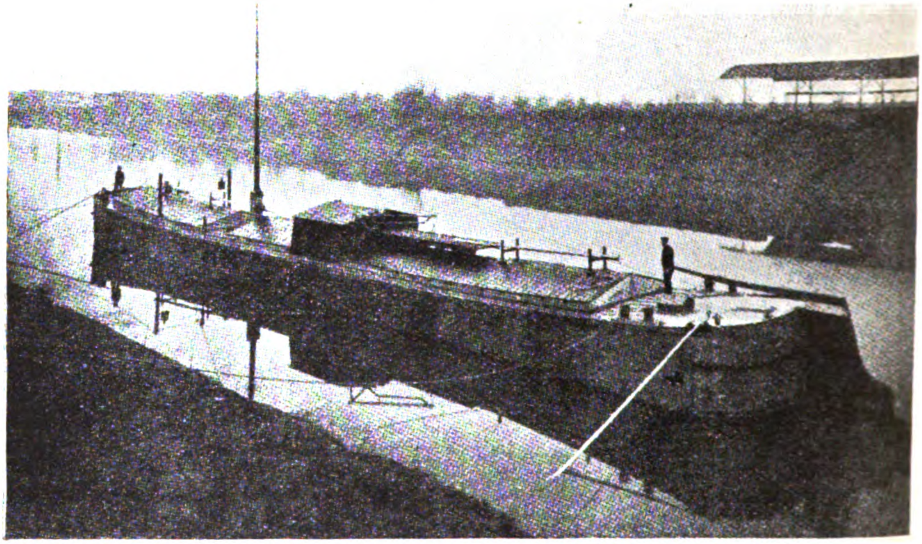


Fig. 1.—Concrete Barge of 244 tons Capacity.

The barge when empty has 22 in. below the water-line, and 6 ft. when loaded. The empty vessel weighs 71 tons and has a capacity of 244 tons. Its maximum length is 130 ft., and maximum width 16 ft. 6 in. The ratio of weight of vessel to load is 1 : 3·4, as compared with 1 : 2 for most concrete vessels, and it is hoped that in constructing other barges of the same type the ratio may be increased to 1 : 4 or even 1 : 4·5.—*Schweizerich Bauzeitung.*

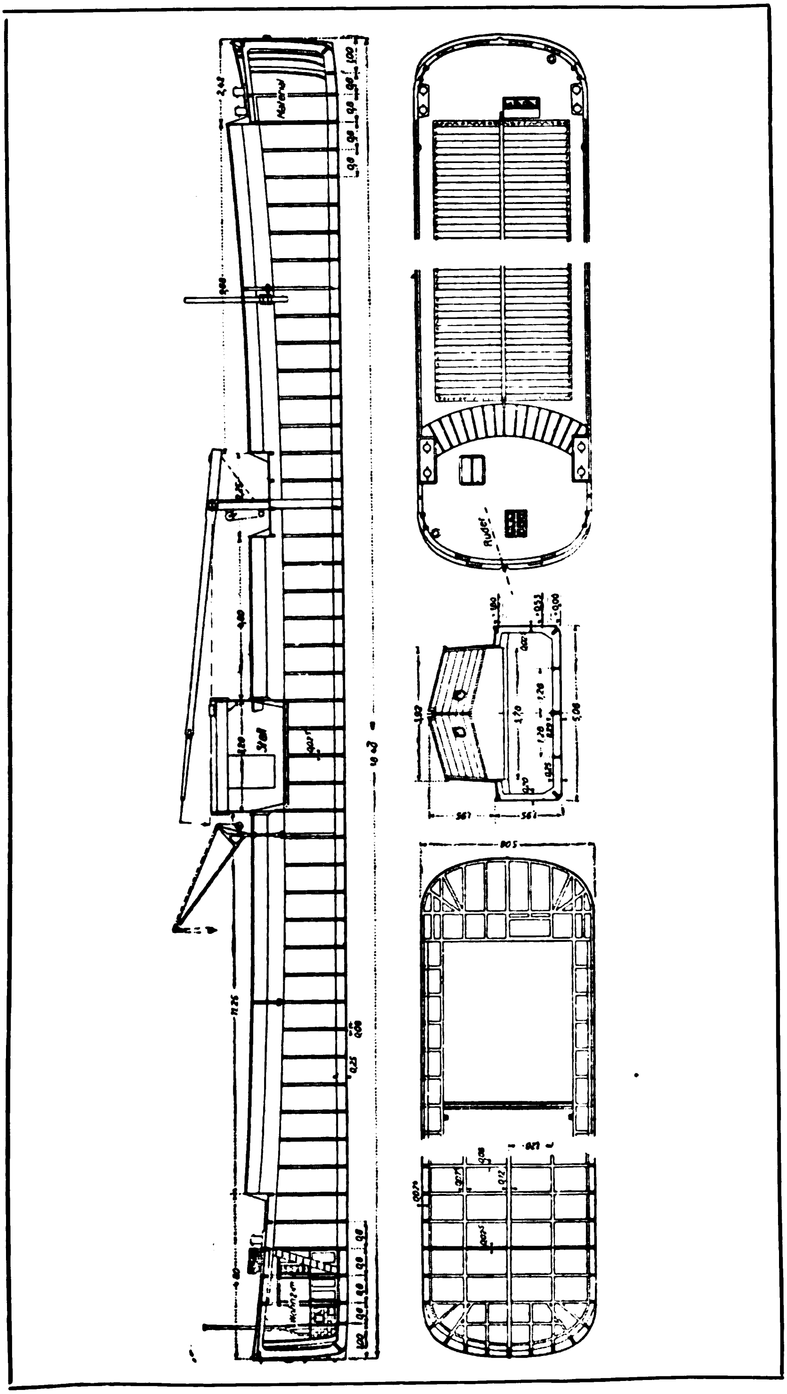
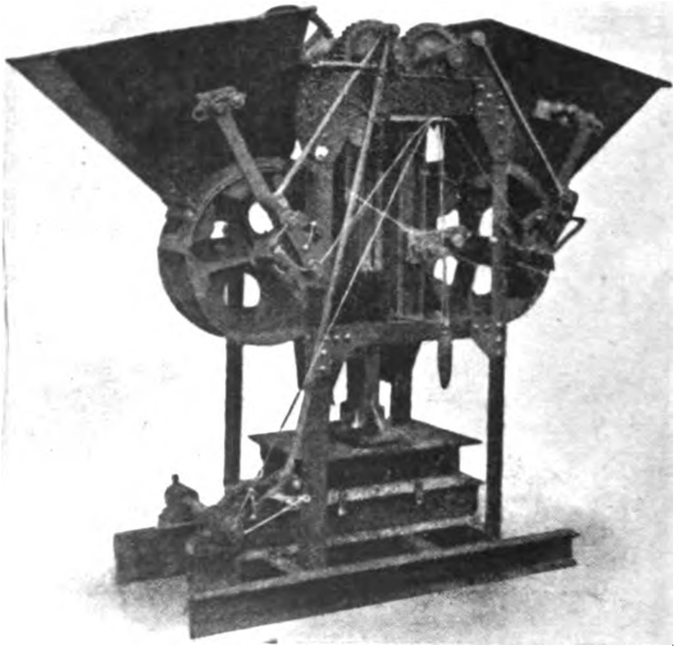


Fig. 2.—DETAILS OF CONCRETE BARGE. (*See p.* 622.)

AN AUTOMATIC RAMMER AND FILLER.

THE Comité des Constructions et Beaux Arts has awarded a special prize to Monsieur L. Jaussand for a machine which he has invented, the Committee feeling that it is a great step in advance of any other appliances now in use for the same purpose. The machine consists of three parts: a concrete distributor, a device imparting a to-and-fro movement to the form or mould, and a rammer or tamper. In the machine shown there are two distributors—one for each half of the mould—each consisting of a hopper with an opening at its lower end closed by a rotating wheel having recesses or "buckets" on its rim. As the wheel rotates each "bucket" is filled in turn, and its contents are later discharged through a chute into the mould. The mould is moved to-and-fro so as to bring the rammer in contact with every part of its contents in turn. The rammer weighs 55 lb., and delivers sixty blows per minute. The machine is suitable for blocks of any thickness up to 16 in. by 12 in. by 10 in., and produces fifty to sixty blocks per hour. The machine weighs about 10 cwt., and requires 1½ h.p. to operate it. Three attendants are needed: one to control the machine, one to remove the blocks as they are made, and one to keep the machine supplied with concrete.



AUTOMATIC CONCRETE RAMMER AND FILLER.

The chief advantages claimed for this machine, as compared with hand-tamping, are saving in labour and rapidity of action. It is sufficiently portable to be readily taken to the building site where the blocks are required.

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Portland Cement	10s. per ton extra		

Boarding for Shuttering—		Sawn.		Wrot.	
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

Sawn Timber for Struts and Supports—	
3 in. by 4 in.	from £24 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £25 ,, ,,

Mild Steel Rods for Reinforcement—		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

MATERIAL AND LABOUR INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

Portland Cement Concrete well mixed with Clean Water in proportion.

1 : 2 : 4—				s.	d.
Do.	do.	in foundation	per yard cube	45	0
Do.	do.	in columns	,, ,,	50	0
Do.	do.	in beams	,, ,,	50	0
Do.	do.	in floor slabs 4 in. thick	per yard super	5	3
Do.	do.	in floor slabs 6 in. thick	,, ,,	7	9
Do.	do.	in floor slabs 9 in. thick	,, ,,	11	3
Do.	do.	in walls 6 in. thick	,, ,,	8	0

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

Steel Reinforcement, including Cutting, Bending and putting into Position and securing with Stout Binding Wire—		s.	d.
From ¼ in. to ⅜ in.	per cwt.	25	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

Shuttering—		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	54	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	45	0
Do. do. in small quantities	per ft. super	0	9
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	10
Do. do. as last in narrow widths	,, ,,	1	0
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	0
Do. do. as last in narrow widths	,, ,,	1	2
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

WAGES.—The rates of wages on which the above prices are based are those current in the London area, namely:—Carpenters and joiners, 1/7½ per hour; Labourers on building works, 1/2¾; Operators on concrete mixers and hoists, 1/3¾.

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

MEMORANDA.

PROSPECTIVE NEW CONCRETE WORK.

Aberdeen. — *Roads.* — The Corporation Streets and Roads Committee contemplates a big scheme of roadmaking involving an expenditure of £267,000.

Barrow.—*Road.*—The Corporation has received sanction to borrow £19,350 for the construction of a new coast road to Bardsea.

Berwick.—*Harbour.*—The Bill of the Corporation of Berwick, for the proposed harbour scheme, which is estimated to cost £100,000, has been passed for the third reading by the Unopposed Bills Committee of the House of Lords.

Birstall.—*Houses.*—The U.D.C. has decided to build 69 more houses on the Nova estate.

Bishopsworth. — *Sewage Works.* — The U.D.C. has applied to the M.H. for sanction to borrow £1,500 for sewage disposal works in the parish.

Bognor.—*Sewage Works and Promenade.* —The U.D.C. has applied to the M.H. for sanction to borrow £5,000 for sewerage and sewage disposal works, and £18,000 for improvement of the roadway and the promenade in front of Lansdowne Place.

Bournemouth.—*Bridge.*—The Corporation will apply to the M.H. for sanction to borrow £1,000 for a new bridge in Boscombe Chine Gardens.

Bridlington.—*Road.*—The R.D.C. has applied to the Unemployment Grants Committee for a grant for the construction of a new road from Chapel Street to High Street.

Chertsey. — *Houses.* — The U.D.C. has approved a lay-out plan for 141 houses.

Chertsey.—*Bridge.*—A bridge to be constructed over the Sunbury watersplash is estimated to cost £3,000.

Chester-le-Street. — *Houses.* — The U.D.C. has applied to the M.H. for sanction to erect 250 more houses on the Council's building estate.

Colchester.—*Quay.*—The T.C. has resolved to apply to the M.H. for sanction to borrow £6,150 for the suggested improvement of Crane Quay and King Edward Quay.

Congleton.—*Gasholder.*—The T.C. has received sanction to borrow £9,500 for a new gasholder.

Coventry.—*Sewage Works.*—The City Council has applied to the M.H. for sanction to borrow £45,000 for the extension of the sewage disposal works at Baginton.

Glasgow.—*Houses.*—The T.C. has decided to invite tenders for the erection of 138 houses in Campbell Street, Springbourn.

Glyncorrwg.—*Culvert.*—The U.D.C. has obtained sanction to construct a new culvert, estimated to cost £2,000.

Greenock.—*Houses.*—The Corporation proposes to erect 150 houses.

Halifax.—*Reservoir.*—The T.C. has under consideration the construction of a new reservoir at Gorple Valley.

Hastings.—*Road.*—The T.C. will submit to the M.H. schemes for the continuation of the new arterial road from Sedlescombe Road to Battle Road, at an estimated cost of £6,043, and the enlarging of Ecclesbourne Reservoir at an estimated cost of £14,800.

Hemsworth.—*Sewage Disposal Works.*—The U.D.C. has applied to the M.H. for sanction to borrow £6,500 for sewage disposal works.

Horsforth.—*Road and Bridge.*—The M.H. has passed the scheme by the U.D.C. for the new Calverly Road and bridge.

Hove.—*Pier.*—A scheme for the erection of a ferro-concrete pier, 1,250 ft. long and estimated to cost £180,000, is nearing completion. The scheme includes a theatre seating 1,400 people, and a dance or concert hall accommodating 1,000 people.

Ilkeston. — *Reservoirs.* —The Ilkeston and Heanor Water Board proposes constructing two reinforced concrete service reservoirs with a capacity of 1,500,000 gallons each.

Kingston-on-Thames. — *Roads.* — The Surrey C.C. has agreed to undertake the construction of the two arterial roads, known as the Kingston and Sutton by-pass roads, at an estimated cost of £567,500.

Kirkcudbright.—*Bridge.*—The C.C. and the T.C. has under consideration a scheme for the erection of a new bridge estimated to cost £33,000.

Marske.—*Road.*—The Gisborough R.D.C. contemplates the continuation of the coast road from Marske towards Saltburn, at an estimated cost of £36,000.

Middlesbrough.—*Docks.*—Mr. A. C. Mitchell has been authorised by the Corporation Ferry Committee to proceed with the scheme for new docks and ferry landing, which is estimated to cost £20,000.

Ormskirk.—*Sewage Farm.*—The U.D.C. has applied to the M.H. for sanction to borrow £6,000 for the purchase of a sewage farm, and for certain works in connection therewith.

Poole.—*Reservoir.*—The T.C. has decided to prepare plans for the construction of a new reservoir at Corfe Hills, with a capacity of 5,000,000 gallons.

Prestolee.—*Sewage Disposal Works.*—The R.D.C. has applied to the M.H. for sanction to borrow £2,355 for sewage disposal works.

Runcorn.—*Reservoir.*—The U.D.C. has applied to the M.H. for sanction to borrow £14,000 for the construction of a reservoir on Runcorn Hill.

St. Helens, I.O.W.—*Road.*—The U.D.C.

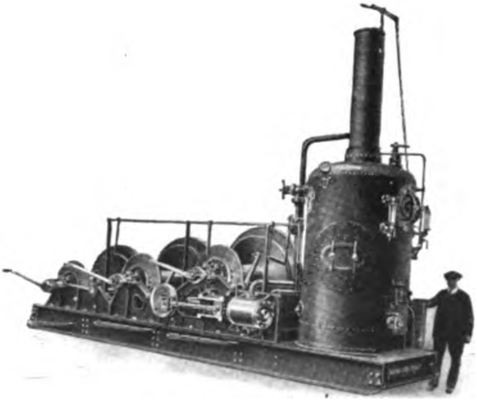
The "Zenith" Friction Winch.

THE B.S.P. PRODUCTS have made their way by the way they're made.

BUILT IN FOUR SIZES,
with one, two
—— or ——
three barrels.

For Steam, Petrol, Paraffin or Electric Drive.

OUR illustration shows the rear view of the operating side of a Treble-Barrelled "Zenith" Winch, with Boiler mounted as a complete unit on steel girder frame.



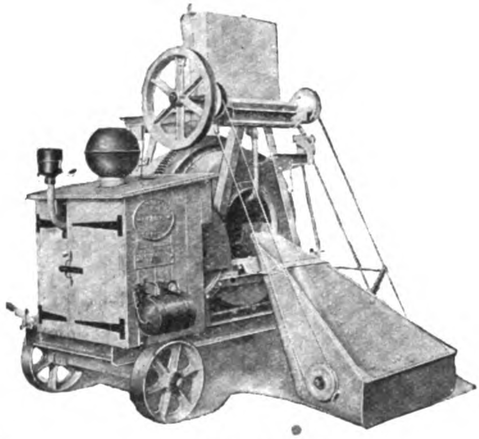
Full details and prices sent on application.

The BRITISH STEEL PILING CO.

DOCK HOUSE, BILLITER STREET, LONDON, E.C.3.

The Victoria CONCRETE MIXER

7 Cubic Feet of MIXED CONCRETE Per Batch



THE VICTORIA MIXER
MODEL No. 7.
PRICE
£317
COMPLETE.

THE latest No. 7 VICTORIA Mixer is just the very machine you have been looking for—a medium-sized mixer—built to last—extremely mobile—easy in operation, and capable of excellent service at high rate of output. Moreover, it is BRITISH made throughout—nothing flimsy in its construction—thoroughly reliable, and at its price it is undoubtedly the finest value for money obtainable.

ALL we ask is that you write for full details of this machine, study the specification and compare the quality—we claim the No. 7 VICTORIA to be the most compact and efficient mixer yet produced, and we are prepared to substantiate our claim and prove to your own satisfaction that here is a British machine of outstanding merit. May we arrange a demonstration?

Why not write for full particulars without delay?

STOTHERT & PITT LTD
MIXER DEPARTMENT
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STOTHERT & PITT LTD
Manufacturers of
CRANES OF ALL TYPES
GRABS · MIXING PLANTS
ROADMAKING MACHINERY
ROTARY PUMPS

has applied to the Unemployment Grants Committee for financial assistance in the construction of a new road from St. John's Church corner to Thornton and Upton.

Saxmundham.—*Bridge.*—The proposal for a reinforced concrete bridge, with a roadway 24 ft. wide and a path 6 ft., will be placed before the M.H.

Smethwick. — *Gas-Holder Tank.* — Included in the scheme for the relief of unemployment is the construction of a concrete gas-holder tank at an estimated cost of £48,800.

Stafford.—*Houses.*—The T.C. has decided to apply to the M.H. for sanction to build 60 houses.

Stainforth.—*Sewage Disposal Works.*—The R.D.C. has applied to the M.H. for sanction to borrow £5,300 for sewage disposal works.

Sunderland.—*Road.*—The making of a new road to the Hendon Beach is included in the relief scheme of the Corporation Highways Committee.

Tilbury.—*Docks.*—The Port of London Authority is embarking on a programme of new dock works and extensions, which, exclusive of land, will cost about £5,000,000.

Turton.—*Sewage Disposal Works.*—The U.D.C. have decided to apply to the M.H. for sanction to borrow £9,000 for extensions and alterations to the Eagley sewage works.

Tutbury. — *Bridge.* — The R.D.C. has passed plans for the new Belmont bridge. The bridge is to be of reinforced concrete, and estimated to cost £950.

Wellingborough. — *Road.* — The U.D.C. proposes constructing a new road from Doddington Road to Northampton Road.

Weymouth.—*Esplanade.*—The Corporation has approved a scheme for widening the esplanade, at an estimated cost of £10,500.

Whitehaven.—*Road.*—The Cumberland C.C. has resolved to apply to the M.H. for sanction to borrow £17,300 for the proposed loop road at Whitehaven.

Witham.—*Sewage.*—The M.H. has held an inquiry into an application by the U.D.C. to borrow £7,000 for sewage disposal works.

Wrexham. — *Street.* — The Corporation propose constructing a new street.

York.—*Bridge.*—The Corporation Streets and Buildings Committee has decided to put forward for the approval of the City Council a scheme for a bridge across the L.N.E.R. from Wiggington Road to Burton Stone Lane, and a new road from Tang Hall to Hull Road.

TENDERS ACCEPTED.

Bournemouth.—*Promenade.*—The T.C. has accepted the tender of Messrs. Anthony Fisher & Son, at £49,989 12*s.*, for the construction of Boscombe promenade and other works.

Eagley.—*Sewage Works.*—The tender of Messrs. J. Byrom, Ltd., Bury, Lancs., for altering and reconstructing the settling tanks, bacteria beds, etc., has been recommended for acceptance.

Frome.—*Bridge.*—The U.D.C. has accepted the tender of Mr. F. J. Seward, Frome, at £250, for the construction of a ferro-concrete bridge over the Rodden Lake Stream.

Haverfordwest.—*Road.*—The T.C. has accepted the tender of Messrs. Watson and Horrocks, Ltd., Bridgend, at £4,066 6*s.* 11*d.*, for laying a ferro-concrete road from Queen's Hotel to Bridgend Square.

Honiton. — *Bridge.* — The T.C. has accepted the tender of Messrs. Fothergill, Bros., Exeter, at £505 7*s.* 3*d.*, for the construction of a reinforced concrete bridge.

Hove.—*Facing.*—The Corporation has accepted the tender of Messrs. Parsons & Sons, Hove, at £1,279, for facing with Portland cement the Hove baths.

Littletown.—*Bridge.*—The U.D.C. has accepted the tender of Messrs. Fothergill Bros., at £505 7*s.* 3*d.*, for the erection of a reinforced concrete bridge.

London.—*Docks.*—The contract for the improvements at Surrey Commercial Docks, part of the London Authority's £5,000,000 unemployment relief works scheme, has been placed with Messrs. Topham, Jones, and Railton, of Westminster.

London. — *Reservoir.* — The Metropolitan Water Board has accepted the tender of The British Reinforced Concrete Engineering Co., at £117 7*s.* 6*d.*, for repairs at Banbury reservoir.

London.—**Greenwich.** — *Coal Bunkers.*—The L.C.C. has accepted the tender of Messrs. E. C. & J. Keay, Ltd., at £980, for the erection of partitions to the coal bunkers at Greenwich power station.

London. — **St. Marylebone.** — *Intake Dock.*—The B.C. has accepted the tender of Messrs. H. Sabey & Co., at £2,539, for the construction of a new intake dock on the Regent Canal.

Mablethorpe.—*Promenade Extensions.*—The U.D.C. has accepted the tender of Mr. Hodgson, at £329, for concrete work in connection with promenade extensions.

Nelson.—*Retaining Wall.*—The Corporation has accepted the tender of Messrs. Simplex Piles, Ltd., for the erection of a reinforced concrete retaining wall.

Newbury.—*Footpath.*—The Corporation has accepted the tender of Messrs. Love & Brown, Newbury, at £367 7*s.* 6*d.*, for surfacing with concrete for a distance of 470 lineal yards in the Enborne Road.

Ollerton, near Newark.—*Bridge.*—The Notts C.C. has accepted the tender of Messrs. Gray's Ferro-Concrete Co., 156, St. Vincent

Street, Glasgow, at £986 3*s*. 11*d*., for the erection of a reinforced concrete bridge over the river Maun at Ollerton.

OXFORD.—*Bridge Widening, etc.*—The T.C. has accepted the tender of Messrs. Stewart & McDonnell, Westminster, at £10,229 2*s*. 6*d*., for the reconstruction and widening of Seacourt Bridge, and widening and deepening the stream at the bridge.

SOUTH SHIELDS.—*Boiler Foundation.*—The T.C. has accepted the tender of Messrs. G. Bailey & Co., at £4,422 18*s*. 10*d*., for foundations to new boilers at the electricity works.

SUNDERLAND.—*Reservoirs.*—The Sunderland and South Shields Water Co. have accepted the tenders of Messrs. Holloway Bros., of London, and Messrs. Brims & Co., of London and Newcastle, for the construction of two new reservoirs, which are to be of concrete throughout.

WALTON.—*Water Works.*—The Metropolitan Water Board has accepted the tender of Messrs. Christiani and Nielsen, at £116,114 19*s*. 2*d*., for the construction of 18 reinforced concrete primary rapid filters, each about 26 ft. by 16 ft. 6 in., etc.

RECENT PATENT APPLICATIONS.

195,585.—J. E. Lanhoffer: Apparatus for calcining of cement.

198,758.—R. Bendall: Construction of roads and other similar work and concrete slabs therefor.

199,056.—S. Rigby: Processes and plant for the manufacture of cement.

199,128.—H. C. Merrill: Concrete walls.

199,545.—L. Santarella: Construction of reinforced concrete trusses.

199,552.—J. H. Walker and R. E. Knight: Construction of the steel reinforcement in reinforced-concrete structures.

199,575.—O. Nickel and R. Markwity: Manufacture of cement.

TRADE NOTICES.

A MAIN road through Reigate to Buckland, which carries fast heavy and light traffic, including traction engines, is now under construction by the Surrey County Council. The width of carriageway is 30 ft., and the total width is 60 ft. between fences. The total thickness of concrete is 8 in., laid in alternate bays in two courses; the bottom course is 6 in. thick of 4 : 2 : 1 concrete, and the top course in proportions of 4 : 2 : 1½. The bottom course is well rammed with punners in and around the wire reinforcement, and when sufficiently advanced the top course is applied immediately so as to give the two layers a simultaneous setting. The reinforcement used is partly Walker-Weston patent interlocked double-layer. The filling varies in places from 3 in. to 7 ft. for a length of about 4,000 ft. The surface is finished to a camber of 1 in 50. After completion the surface is covered to a depth of 3 in. with soil and kept damp for 28 days, and is then uncovered and thrown open to traffic. No sign of cracks has so far been observed. It is interesting to note that the materials required for the construction have been hauled over the finished sections of the road by a 16 ton traction engine with three trucks of an approximate weight of 45 to 50 tons over half bays measuring 18 ft. by 15 ft., without the surface or the joints being affected.

The reinforced concrete road in Cambridge referred to in our last issue is being carried out to the specification of the British Reinforced Concrete Engineering Co., Ltd., of Manchester and London.

The Expanded Metal Company, Ltd., of York Mansion, Petty France, Westminster, S.W.1, have issued a pamphlet of 175 pages fully illustrating and describing the production, properties and applications of their "Expamet" expanded metal, "BB" expanded metal lathing, and "Exmet" reinforcement. "Expamet" expanded metal is rolled sheet metal—steel, brass, copper, etc.—cut and expanded into meshes of various shapes and sizes; each size of mesh is made in several weights, for, while a particular mesh remains constant in size, centre to centre, it can be produced from several thicknesses of sheets or plates, and strands of various widths can be made. "Expamet" expanded steel, which is made from mild steel sheets and plates, is specially designed for reinforcing concrete in roads, foundations, walls, floors, roofs, etc. "B B" and "Expamet" lathings are made from mild steel sheets to form a key for plaster in ceilings, steelwork encasing, solid and hollow partitions, exterior walls and other plasterwork. "Exmet" reinforcement is designed particularly as a reinforcement for brickwork, concrete-block work, partition slabs, asphalte, etc. It is manufactured from mild steel of 20, 22 and 24 g., in 2½ in., 7 in. and 12 in. widths and is supplied in standard coils containing 270 or 75 ft. lineal.

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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

OCTOBER 1923 VOL. XVIII. No. 10

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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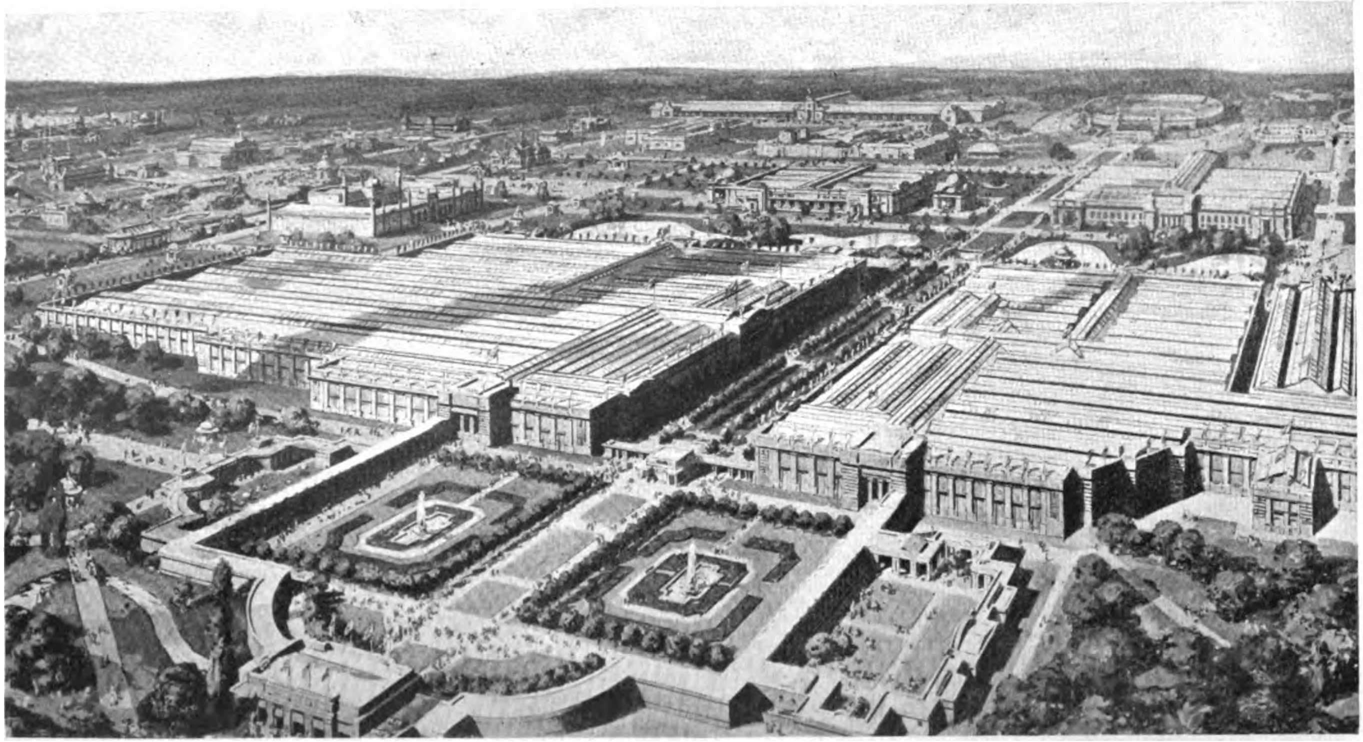

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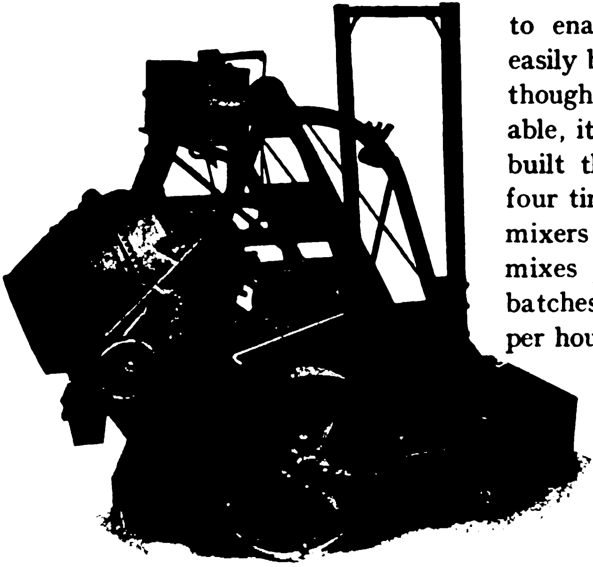

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CONCRETE
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CONSTRUCTIONAL ENGINEERING

Volume XVIII. No. 10. LONDON, OCTOBER, 1923.

EDITORIAL NOTES.

QUICK-HARDENING CEMENT.

THE advent of the new aluminous cement referred to in our August issue has stimulated interest in the possibility of improving the quick-hardening properties of Portland cement. There is no doubt that cement users would gladly adopt a quick-hardening Portland cement in preference to a new cement if the hardening properties could be improved to some degree approaching the latter.

A good deal has been done in recent years by refinements in manufacture, such as fine grinding of raw materials and finished cement, to improve the strength of Portland cement at ages of twenty-four and forty-eight hours, the strengths at these early periods being improved in about the same proportion as the recognised 7-day tests have been improved. But there is still room for further improvement, which would be welcomed by builders and contractors whose operations would be greatly assisted if concrete attained in, say, twenty-four hours the strength usually developed in seven days. Portland cement manufacturers will doubtless be alive to the importance of this matter, and will spare no effort to develop the qualities of cement in this direction. It must be admitted that the British Standard Specification is almost a deterrent to such an improvement, because of the growth in strength that is demanded for neat briquettes between the ages of seven and twenty-eight days.

The strength of Portland cement is due to the presence of certain definite chemical compounds, and there is probably a definite limit to the tensile and compressive strengths that cement may develop. If by fine grinding the strength is developed at an early period it is irrational to demand any substantial increase at later dates, especially in the case of neat cements. The British Standard Specification recognises this to some extent by the use of a formula which reduces the required growth from seven to twenty-eight days as the 7-day test increases; but the allowance is not sufficiently effective, and with the personal element in testing tending to cause irregularities there are risks when efforts are made to produce additional strength at early dates that the balance of the strength available in the neat cement will not suffice to yield the increase demanded by the Standard Specification. It must be remembered that the Specification by its stringent tests for soundness and chemical composition already provides safeguards for the ultimate stability of the cement, so that a cement which otherwise complies with the Specification cannot be classed as unreliable if the neat test fails to show the specified increase at twenty-eight days. Hence, as we have already stated, the British Standard Specification is in some degree a deterrent to the development of quick-hardening cements.

In addition, however, to the measure of quick-hardening properties that can be developed by refinements in manufacture there are the same qualities possessed in a superlative degree by some cements as a result of chemical composition. It is known that cements in which the chemical analysis shows the proportion of alumina to be 2 or 3 per cent. above the average are usually considerably better—other conditions being equal—in their early hardening properties than the average cement. Some brands have gained a high reputation among users as the result of their high alumina content endowing the cement with quick-hardening properties. Such cements are seldom if ever the subject of complaint of delayed hardening or failure to set. It is a striking fact that the orthodox Specification tests do not reveal any superiority in quality, and at seven days the tensile and compressive strengths of such quick-hardening Portland cements are no better than the average cement. This has often been a cause of mystification to the user of cement, who observes that while the Specification tests of two cements may be similar yet one cement may " harden off " much more rapidly than another, and the impression left on the minds of the uninitiated is that tests are of little value.

In the great majority of cases the cement manufacturer is unable to increase the proportion of alumina in his cement at will. The proportion of alumina in the raw materials is usually practically constant in any one deposit, and there is no possibility of increasing this constituent other than by transport of a special raw material at prohibitive cost. It is therefore not without value to the user of Portland cement that the appearance of a rival cement in the market may stimulate research and possibly lead to the production of Portland cement with quick-hardening qualities.

HEAT TRANSMISSION THROUGH BUILDING MATERIALS.

In 1921 the Building Research Board published an extract from the report of the Building Materials Research Committee entitled, " The Transmission of Heat and Gases through, and Condensation on the Surface of, Wall Materials," in which were given the results of tests carried out by Mr. A. H. Barker, Wh.Sc., B.A., B.Sc., of University College. A week or two ago the Board published another report, entitled, " Heat Transmission through Walls, Concretes, and Plasters," containing the results of experiments carried out at the National Physical Laboratory by Mr. Ezer Griffiths, D.Sc. Although dealing with the same subject, the conclusions arrived at in the two reports are widely different. Mr. Barker tested eleven different types of walling such as are commonly used for cottage construction, and found that " the 10-in. concrete cavity unventilated wall with the inner partition of coke breeze offers the greatest resistance to the flow of heat " ; and that this type of wall " doubtless partly owes its efficiency as a heat insulator to the porous nature of the breeze, as it is, of course, a well-known fact that good insulators such as cork, slag wool, or wood owe their insulating properties to the large number of small air-spaces contained within their mass." The materials tested by Dr. Griffiths covered fifteen different types of walling, and in an introduction to the report the Director of Building Research suggests that these tests indicate that " ordinary cement concrete does not show up well; it is better when made with coke breeze aggregate, but walls recently used in building schemes are not thick enough to be effective. In this connection it is instructive to note that three compound concrete walls, including coke breeze, made to the pattern of three well-known special building systems, are each much worse than a solid wall of London stocks in cement, or of Fletton bricks in lime, while they would hardly be comparable at all with a hollow 11 in. wall of London stocks."

In the face of these entirely different results of tests carried out by well-known authorities for the same Department within a period of two years it is necessary in the interests of concrete construction to look further into the matter, and to find the reasons for these opposite conclusions. And the reason is not far to seek. The tests in 1921 by Mr. Barker compared a 4½-in. double brick wall containing a 3-in. cavity with a concrete wall formed of an external wall of 4 in. of ballast concrete and a 4-in. coke breeze interior wall separated by a 2-in. cavity. Thus like is compared with like. In the more recent tests, however, in which concrete gave such poor results, we find that, whereas the brick wall tested was 9 in. thick, the actual thickness of the material forming the three concrete walls was only 4 in., 5 in., and 6 in.; the 4-in. and 5-in. walls consisted of two leaves with a cavity between, while the 6-in. wall had a vertical sheet of bitumen embedded in its centre. It will thus be seen that three methods of concrete walling, all devised during the post-war housing shortage in order to provide cheap and rapid methods of construction to meet a national emergency, are compared with the current practice in brickwork which is the outcome of experience during hundreds of years. It must therefore be pointed out that concrete block construction is itself developing on recognised lines, and that the systems selected for the latest tests are not those on which the majority of concrete cottages are being built to-day. Modern practice in the construction of cottages of concrete blocks is to use two walls, the outside wall composed of a dense concrete 3 in. or 4 in. thick and the inner wall of porous concrete 2 in. or 3 in. thick, with a 2-in. cavity between. As this is the type of walling now most generally adopted it might well have been included in the tests, and had it been included it would undoubtedly have shown heat transmission resisting properties equal, if not surpassing, those of a 9-in. brick wall. We have it, in fact, on the authority of the tests carried out by Mr. Barker in 1921 that concrete walls of this type with a total thickness of 10 in. (4-in. ballast concrete outside wall, 2-in. cavity and 4-in. breeze inside wall) prevent the transmission of heat better than a 12-in. brick wall (4½-in. double brick with 3-in. cavity).

We submit that the later report cannot be taken as a comparison between current practice in brick and concrete cottage construction; what it does prove, however, is that the methods now generally adopted for concrete walling for small buildings are better in their heat-insulating properties than the special systems tested by Dr. Griffiths, while at the same time the conclusion arrived at by Mr. Barker that the usual types of concrete walls possess superior heat-insulating properties to usual types of brick walls still holds good.

THE EARTHQUAKE IN JAPAN.

It is too early yet to give any definite news as to the actual number of lives lost and damage done by the recent earthquake and subsequent fires in Japan, but the newspaper reports make it clear that the catastrophe is of a magnitude greater than any of its kind recorded in history. It seems, however, that the Government is already preparing plans for rebuilding the stricken areas. In this they are confronted with a difficult task, and one which demands the most serious consideration. The actual building operations will be on a vast scale, but before any attempt is made to replace the permanent buildings a decision will have to be reached as to the methods of design and materials of construction to be employed. In a region subject to earthquakes safety in the future is as imperative a consideration as the rapid provision of shelter for those rendered homeless and of business accommodation. We do not doubt that the Japanese Government and others responsible are making the fullest investigations into the behaviour of various types of buildings, and the damage they suffered both from shock and fire, and it is to be hoped that the results of any such investigations will be made public for the benefit and guidance of other districts liable to these disturbances of the earth's crust.

Detailed accounts which have reached this country of the effect on buildings are so far very, very meagre, and we have only seen references to this aspect from correspondents of *The Times* and the *Morning Post*. The former said: " Some concrete buildings stood the shock well, but most succumbed afterwards to fire "; and in a later report: " Most of the high concrete buildings in the earthquakes show fissures in the third-floor façades, but above and below that floor little damage, so that it is obvious where the strain comes." The *Morning Post* correspondent says: " Modern buildings of steel and concrete stood the shock and resisted fire, while the buildings of native construction were quickly destroyed." No definite conclusions can be drawn from such brief statements, but it seems that the experience of San Francisco, and Valparaiso, and more recently of Chile, has been confirmed, and that concrete buildings were the most successful in withstanding the shocks.

We take this opportunity of expressing our sympathy with the Japanese nation in the great loss of life and property they have sustained, and would add that if any of our readers in that country would care to send us any particulars as to the behaviour of the different types of structures during the shock and fires we should be pleased to receive them for publication.

WEARING PROPERTIES OF CONCRETE ROADS.

A striking commentary on the esteem in which concrete roads are held in America was made last month by Mr. W. C. Sproul, a member of the United States Senate for twenty-two years, and ex-Governor of Pennsylvania. In an interview with a Press representative he said: " As a result of my experience I have come to the conclusion that macadam roads are of little use in face of the tremendous volume of traffic which main roads are now required to carry. Concrete is the only material which is of any use. We [in Pennsylvania] are making all our roads of concrete, and we find that once the initial cost has been overcome they can be maintained for 150 dollars per mile per year compared with 1,200 dollars to 2,500 dollars per mile per year under old conditions. In addition to that, the old roads required rebuilding every two or three years, whereas the concrete-surfaced road will stand for a long time." He also mentioned that Pennsylvania has taken steps to prevent the roads being destroyed by very heavy traffic by limiting the weight of any vehicle to 26,000 lb.

Those responsible for municipal government in this country are no less urgent in their desire to have the best roads obtainable. This desire was last month emphasised by Alderman Carden, of Brighton Town Council, when he said that "To cope with motor traffic it will be necessary to make a concrete road from London to Brighton." He also pointed out the benefits which would result if such necessary works were put in hand this winter in order to relieve unemployment. "The Government," he said, "should stop the dole and pay for the widening of our roads," an opinion which all concerned with the well-being of the country will heartily endorse.

To those Borough Surveyors and Engineers who have not had personal experience of concrete roads, the following extract from a report by Mr. W. Lionel Jenkins, M.A., M.Inst.C.E., F.S.I., Borough and Waterworks Engineer of Newport, should be of considerable interest. "This road" [the Newport–Cardiff main road], he writes, "carries very heavy traffic, being situated between the ports of Newport and Cardiff. All traffic to South Wales from the south of England also uses this road. In 1919, owing to the difficulty of maintaining the surface of the road in a satisfactory condition, it was decided to lay a trial length of reinforced concrete roadway just within the Borough boundary. The work was carried out by the Corporation with workmen obtained from the Labour Exchange. The concrete consists of local limestone, Bideford gravel and Portland cement, six inches in thickness, reinforced with No. 9 B.R.C. fabric (single layer). . . . One-half of the width of the roadway was laid at a time, and the traffic maintained without interference. After carrying the heavy traffic for four years this portion of the roadway is in an excellent condition and, apart from the cost of tar-spraying, the cost of upkeep has been *nil*." This report is but one of many we have received which prove that all-concrete roads laid under careful supervision and with proper materials are the most durable of all modern methods of road laying.

CONCRETE AND THE COLLEGES.

THIS being the time of the year when young engineers' thoughts turn to colleges we may raise again the question of engineering education, with particular reference to concrete construction. In this subject we find two opposed camps: on the one hand are the ultra-practical people who hold the opinion that the college-trained man is full of theoretical notions which are of very little value, and on the other side we have the University people bewailing the fact that commercial firms do not properly appreciate the value of scientific theory and method. Both parties are right and both are to blame, and the industry as a whole suffers from the lack of co-ordination between the two.

In the first place we ought to realise that college training cannot take the place of practical experience unless our colleges make a fundamental change in their methods, and this seems to us to be improbable. The colleges can and do give a thorough training in the theory of a subject, and can only give such practical and experimental training as is necessary to explain the theory. At the same time we think the theoretical treatment given is often too mathematical and insufficiently in touch with modern conditions. We believe, for instance, that the theory of reinforced concrete as employed by practising engineers is practically never given to students in the colleges of University rank, with the result that students graduating from the colleges have a false idea of the manner in which the calculations are actually made in practice.

On the other hand many students, particularly those who have started their practical training and are attending special courses of lectures on the theory of the subject, are too anxious to run before they can walk. If young engineers are to have that complete grasp of the subject which is so desirable they must understand thoroughly the underlying principles of all structural mechanics, and they can do that only by building up the subject by logical steps from one point to another; the work is laborious, but, if construction is the breath of life to us, it is full of interest, and here it is that the personality of the lecturer can make or mar the work. It is often found that the engineering professor is a man of high scientific attainments but without that mystic power of transmission which characterises the real teacher. The governing bodies of the Universities should remember that the primary function of a professor is to instruct his students rather than to shed glory on his college by the brilliance of his academic achievements.

With regard to concrete construction there is much that the colleges may do to rear the rising generation of engineers in the traditions of its already great achievements and to instil in them the faith in its boundless possibilities.

CONCRETE HOUSES FOR EDINBURGH.

THERE are already signs that concrete will play an important part in the activities of local authorities and private enterprise under the new Housing Act. Following the decision of the Belfast Corporation to build several hundred concrete-block houses, the Edinburgh Town Council has adopted a report of its Housing and Town-Planning Committee to use concrete for the erection of its new houses. This decision was arrived at after a deputation had visited the concrete housing schemes in Newcastle and Leeds, and had expressed themselves as being entirely satisfied with the appearance and structural soundness of these houses. The system to be adopted is the "Duo-Slab" system, which consists of slabs 4 ft. by 3 ft. fitted into grooves in concrete piers spaced 4 ft. apart. The walls are formed of an outer slab and an inner slab (each 3 in. thick) with a 2-in. cavity. The wall faces are roughcasted or finished with cement on the outside and plastered internally. The report states: "In strength it [the system] is certainly of adequate substance for buildings of two stories in height, and it could be strengthened for the erection of three-story tenements. It is impenetratable to moisture and there is no appearance of condensation."

WEMBLEY REVISITED.

A RECENT writer in *The Times* conjured up a vision of Macaulay's New Zealander, transferred from London Bridge to one of the heights around Wembley, sketching the ruins of the British Empire Exhibition instead of St. Paul's. Perhaps he will do so, but much water will have passed under London Bridge and many millions will have seen the Exhibition before that opportunity occurs. For the buildings at Wembley are intended to endure, if not for all time, at least until most other buildings have become fit subjects for the New Zealander's pencil. Perhaps no greater contrast to the usual type of exhibition building could be found than the structures now rapidly taking form at Wembley. Except for the Indian Pavilion, which is to be decorated with plaster domes and minarets, no temporary materials are used anywhere. All is concrete and steel, the few bricks to be seen only serving to emphasise the overwhelming manner in which the newer materials have found favour with the architects and engineers responsible. The requirements called for were strength, durability, and speed of erection, and the realisation that these were best met by concrete and steel has resulted in these materials being almost exclusively adopted.

A year ago the site, covering some 130 acres, was open country except for the stadium nearing completion and the plant and litter in connection with the erection of that structure which were lying about. Now about half of the site is covered with buildings either roofed in or in course of roofing. The Stadium is seen in its setting with the other huge buildings now rising up around it, and although it loses some of the enormous scale it seemed to possess when standing alone, it none the less forms a noble background to the vista along the tree-lined main avenue when seen in the perspective intended by the designers.

As will be seen from the aerial perspective in our Frontispiece, the main entrance opens on to a formal garden with fountains, through which the main avenue, with the Palace of Industry and the Machinery Hall on either side, leads to the Stadium. Entrances to these buildings are placed on the main avenue, but there are alternative covered ways leading to them from the entrance. These covered ways, formed of concrete columns in pairs tied at the top with reinforced concrete beams, are seen in the illustration on either side of the garden. The buildings to the left and right of the main avenue are the Palace of Industry and Machinery Hall respectively, the largest buildings in the Exhibition.

Some idea of the vast extent of the Palace of Industry and the Machinery Hall may be gained from *Figs.* 2 and 3. As will be seen, the structures are entirely of concrete and steel, with the roof partly glazed and partly filled in with asbestos cement sheeting. The concrete columns were poured *in situ*, while the curved members forming the arches and supporting the roof were cast on the floor of the building and hoisted into position. The floor is a reinforced concrete raft extending over the whole of the area. The roof trusses of the main aisles seen in the middle of the illustrations are of steel, but for all the subsidiary aisles pre-cast reinforced concrete trusses are used; these may be seen to the right of *Fig.* 2. All internal partitions and curtain walls are of breeze blocks with a thin face of a mixture of cement and sand. These are made by hand in wooden moulds on the floor of the building (see *Fig.* 3), and stacked until required.

Figs. 4 and 5 are two progress views of the main entrance to the Palace of Industry taken at an interval of one month, and show the methods of construction adopted. *Fig.* 5 also shows three of the concrete flag-masts, a great number of which are employed on different buildings. The weight of the mast is taken on the lion's head corbelled out from the face of the wall and they are kept upright by passing through a ring bonded into the wall about 4 ft. above the lion's head; both the lions' heads and the stays are cast in concrete, and are remarkably clear-cut specimens of moulding. The pillars here, as elsewhere, are cast *in situ* and covered with thin slabs of concrete with a smooth face.

The Exhibition is due to be officially opened in April next, but there is still a great deal to do—some of the pavilions are not yet even commenced. To those who do not realise the possibilities of rapid construction with concrete the task must seem an impossible one, but the authorities have no doubt that everything will be completed before the date fixed for the opening ceremony.

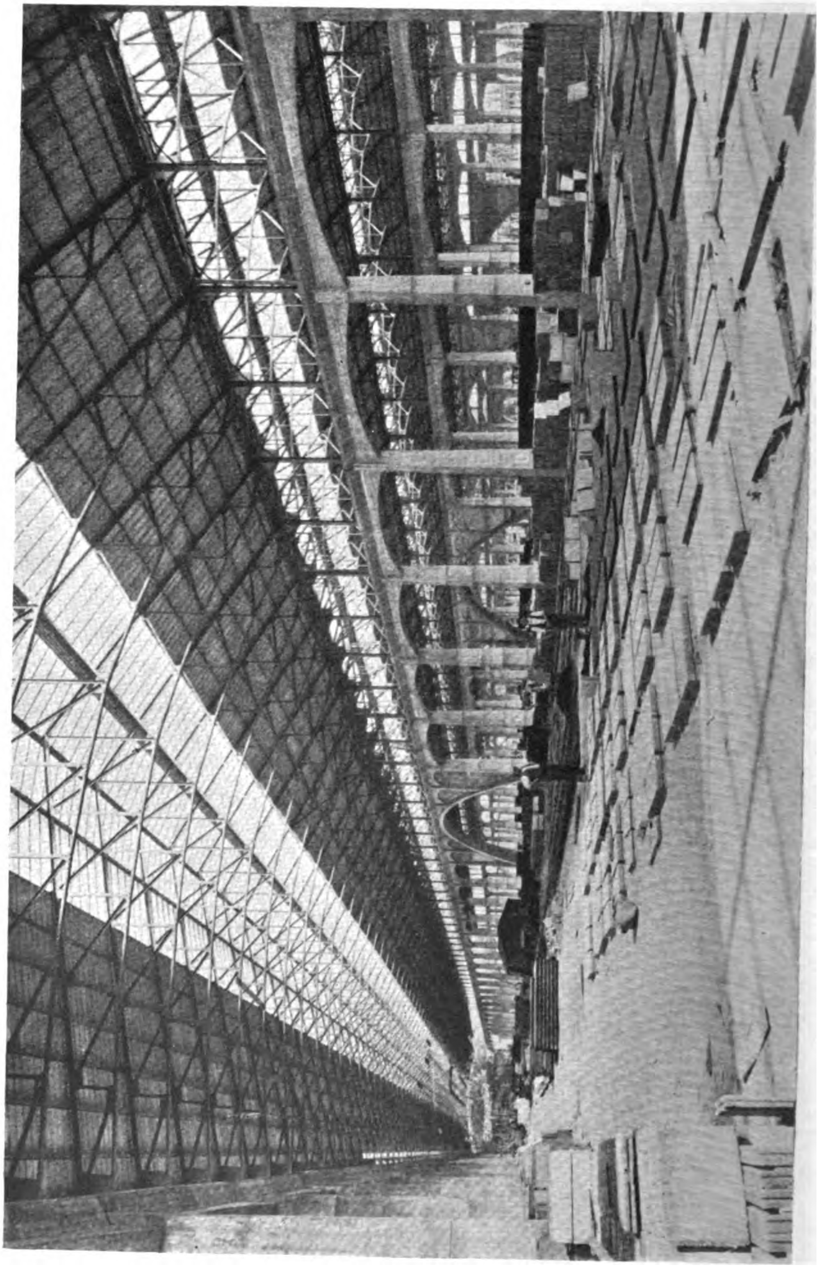


FIG. 2.—BRITISH EMPIRE EXHIBITION, WEMBLEY: MACHINERY HALL. (*See p.* 637.)

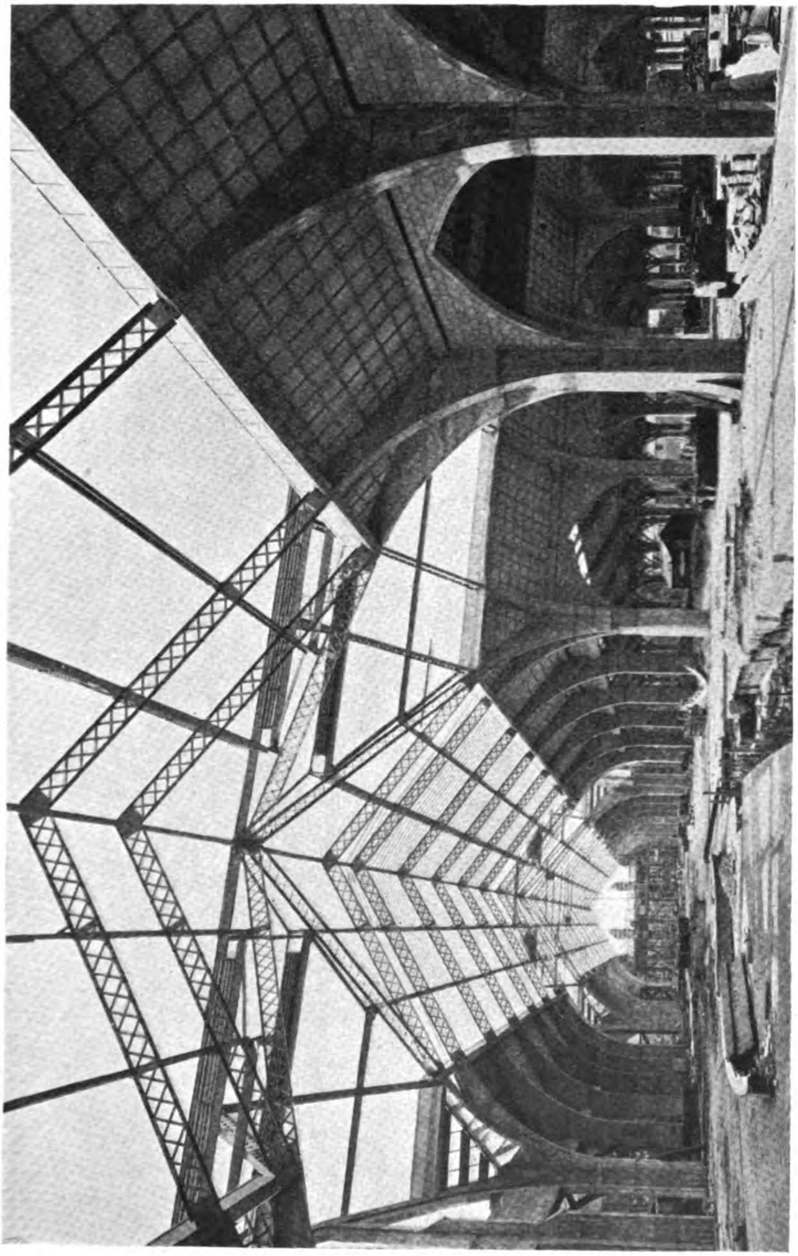


Fig. 3—BRITISH EMPIRE EXHIBITION, WEMBLEY: THE PALACE OF INDUSTRY. (*See p.* 637.)

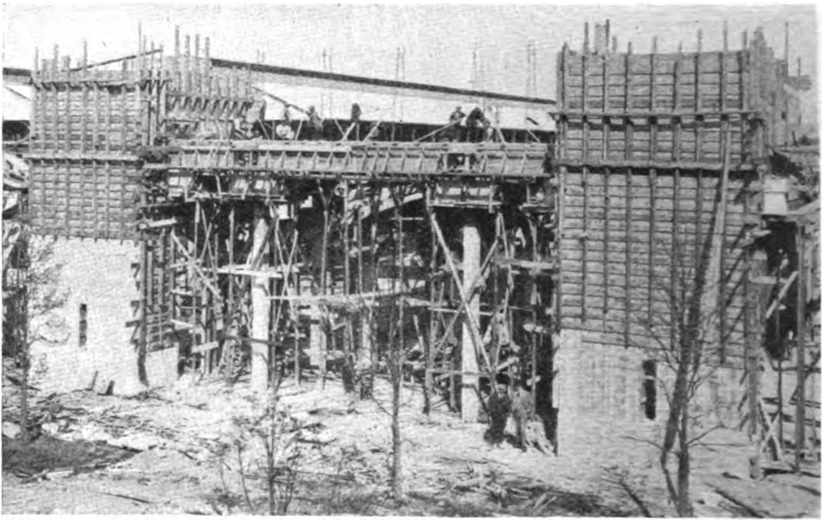


FIG. 4.—PHOTOGRAPH TAKEN ON AUGUST 3.

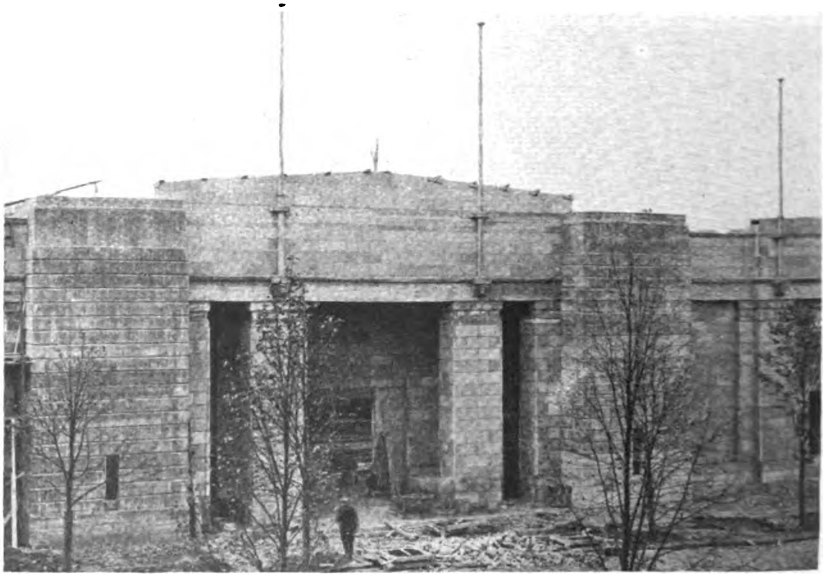


FIG. 5.—PHOTOGRAPH TAKEN ON SEPTEMBER 3.

BRITISH EMPIRE EXHIBITION, WEMBLEY: PROGRESS VIEWS DURING CONSTRUCTION OF MAIN ENTRANCE TO MACHINERY HALL. (*See p.* 637.)

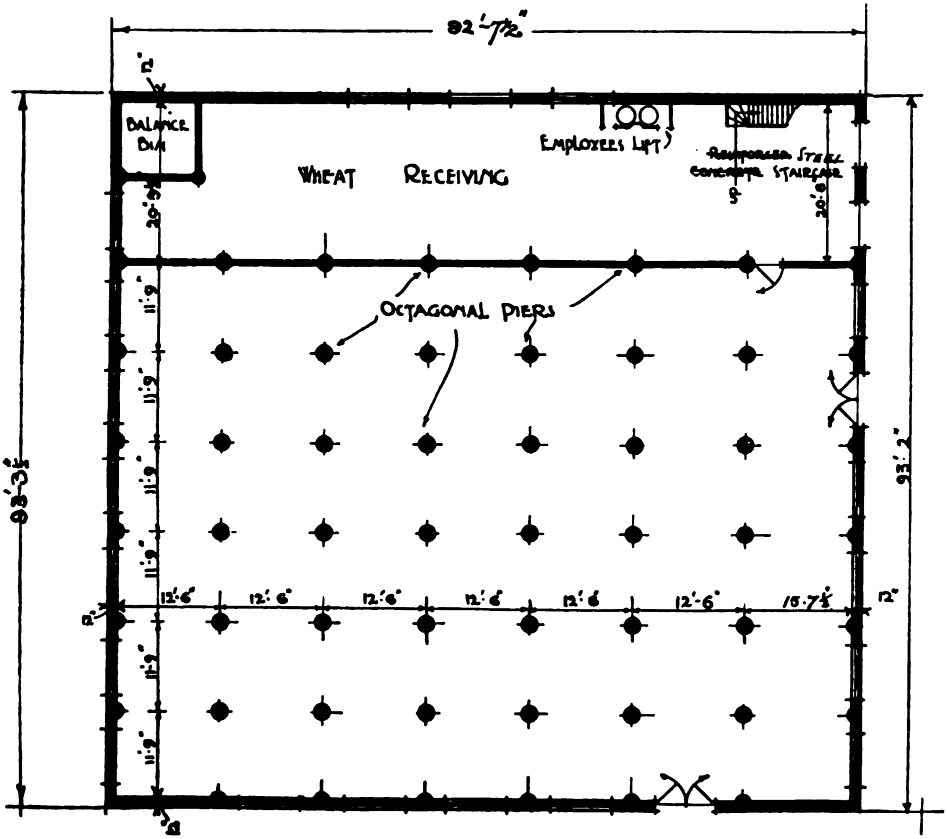
REINFORCED CONCRETE GRAIN SILOS AT AVONMOUTH.

These silos were erected by the Co-operative Wholesale Society's Building Department, London, to the design of their architect for the Southern District (Mr. L. G. Ekins, F.R.I.B.A.).

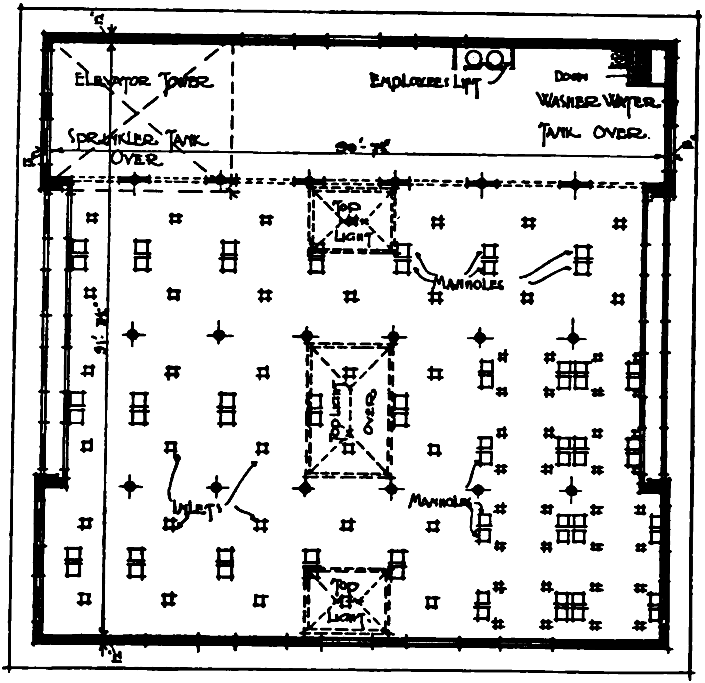
The finished building will afford a fine example of what can be accomplished with reinforced concrete when combined with good architectural design. It is practically square on plan, the dimensions being 93 ft. 3½ in. by 92 ft. 7 in., and has a storage capacity of 447,000 bushels (13,000 tons) of grain. The height from ground level to roof is 132 ft. 6 in., and above this a tower extends for a further 16 ft., making the total height 148 ft. 6 in. A sprinkler tank of 20,000 gallons capacity is contained in the tower. The height of this building will be realised when it is remembered that the height of the Nelson Column in Trafalgar Square is 150 ft.

All grain is moved by belt conveyors and elevators, supplied by Messrs. Spencer & Co., of Melksham, Wilts; the milling plant is being supplied by Messrs. Robinson & Sons, Rochdale.

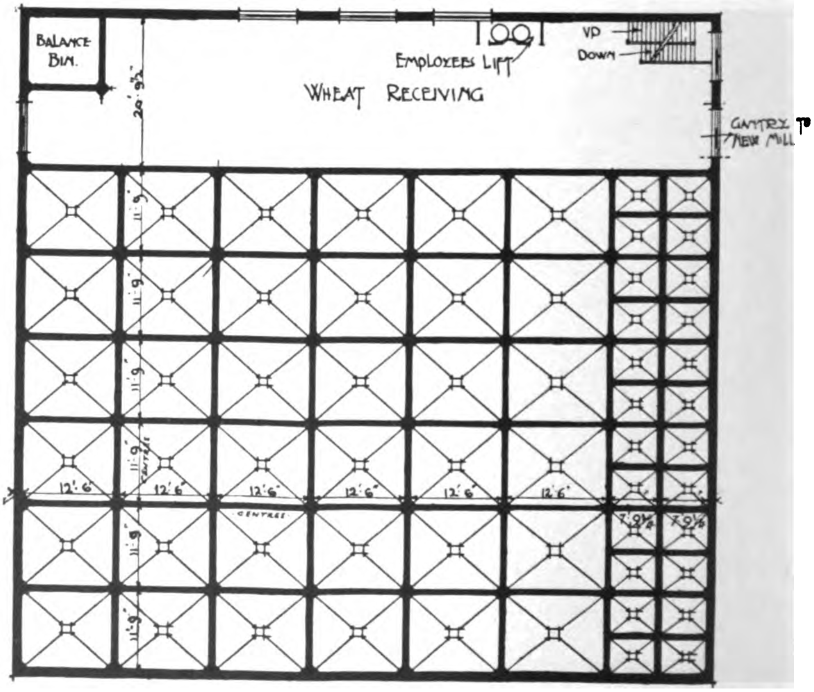
As the ground was too soft to carry the loads required, all the foundations were carried by 14 in. by 14 in. by 55 ft. reinforced concrete piles, which were driven to a set of from ½ in. to 1½ in. in ten blows, and were calculated to have a carrying capacity of 50 tons per pile. In



Reinforced Concrete Grain Silo at Avonmouth: Ground Plan.



Plan through Top Bandway.



Plan through Silos.
REINFORCED CONCRETE GRAIN SILOS AT AVONMOUTH.

all 575 of these piles were driven, in groups varying from 4 to 12. The soft nature of the ground will be evident from the fact that the result of the pile-driving was to raise the level of the site and adjoining land by 2 ft. Very little difficulty was experienced in the driving, as all piles took their set at practically the same depth. The heads of the piles were broken up and a reinforced concrete cap cast round each group to form a base for the reinforced concrete columns supporting the structure. The space between the caps is covered by a 6 in. reinforced concrete floor.

All interior columns are octagonal to bin-bottom level, whilst the face of the external columns project beyond the external walls in conformity with the architect's requirements. It was decided to make the underside of the bin-bottoms level so as to minimise the cost of centering.

The bins are 60 in number, the dimensions of 36 being 11 ft. 9 in. by 12 ft. 6 in., centre to centre of walls, while the remaining 24 are 5 ft. 10½ in. by 7 ft. 8¼ in. centre to centre. The walls of the larger bins are 7 in. thick throughout, and those of the smaller bins 5 in. Each bin is 100 ft. in depth from top to bottom.

Wooden centering was used throughout, the forms being raised as the work proceeded. All mixing of concrete was done by mechanical means, and the mixture was hoisted up a steel tower and fed by gravity to the position required. The concrete for the columns below the bins is a 1 : 1 : 2 mixture, while a 1 : 2 : 4 mixture was used for the columns above the bottoms of the bins.

The bin tops are covered with reinforced concrete slabs 6 in. thick, which form the bandway floor. An opening 2 ft. square is left in the top of each large bin, and 1 ft. 6 in. square in each small bin. Belt conveyors carry the grain over the floor to the bins.

This floor is covered by a reinforced concrete roof which extends over the whole building, light being provided by three large lantern lights. A covering of asphalte is given to the roof, and falls are obtained by varying the height of columns as necessary.

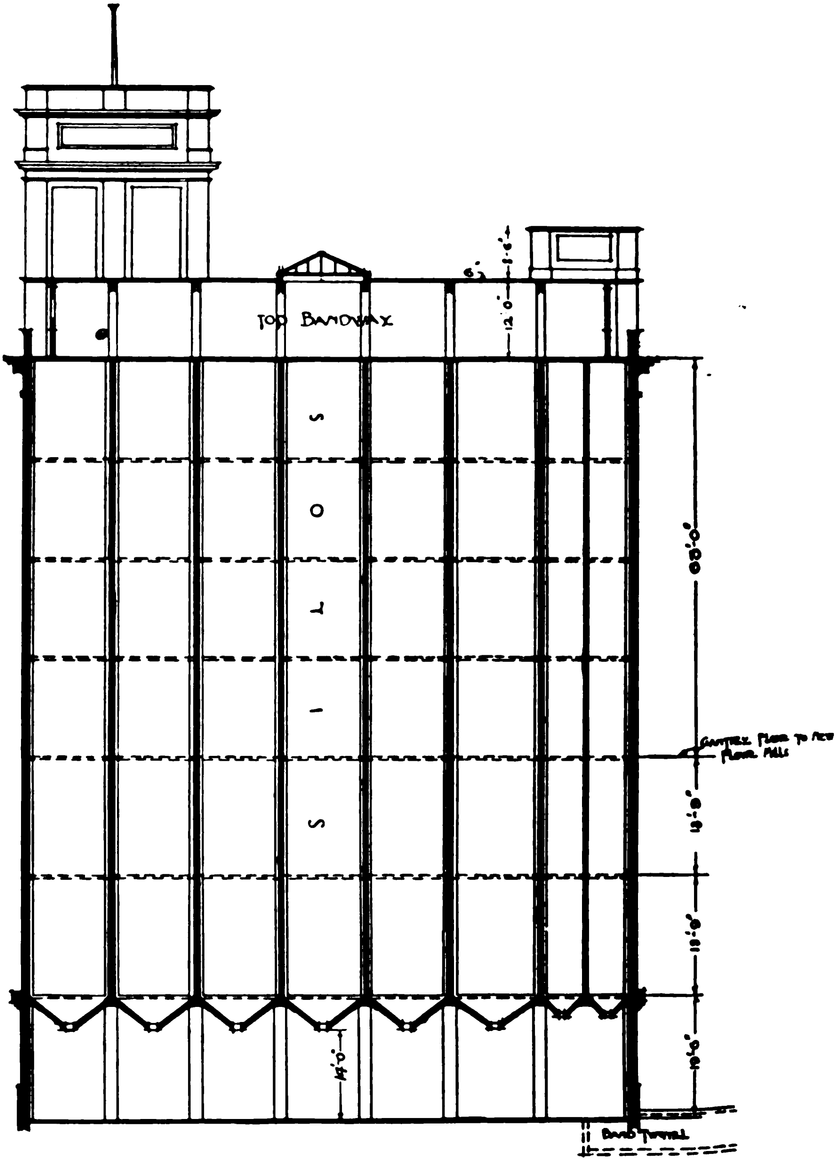
The east end of the building holds the elevators and milling machinery, carried on reinforced concrete floors. Elevator rooms are provided at the north-east and south-east ends of the building to carry the hoisting machinery, whilst a large balance bin occupies the north-east corner from the ground level to the fourth-floor level. A smaller balance bin occupies the south-east corner from the fourth to the bandway floor level.

External walls are in reinforced concrete 7 in. thick, panelled on the external face. The wall surfaces are finished without any skimming coat of cement and sand, and are left as from the shuttering. Steel window sashes are provided throughout, these being supplied by Messrs. Hy. Hope & Sons, of Birmingham.

All steel is to British Standard Specification, and consists of round bars of mild steel.

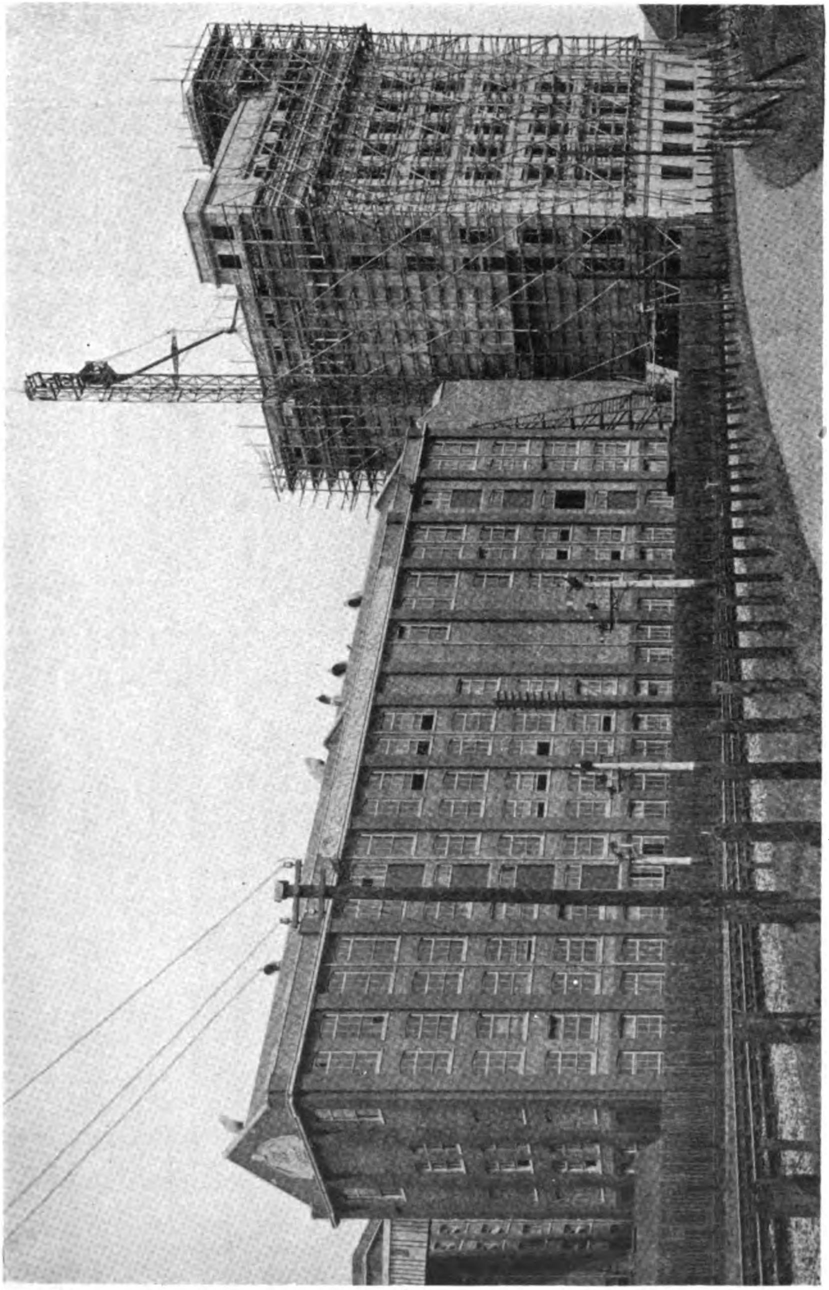
A tunnel extends from the railway siding to the underside of the elevators to carry the belt conveyor, which conveys grain from trucks to the bins. This tunnel is 229 ft. 2 in. in length, 7 ft. wide, and 7 ft. 3 in. in height, and is supported by 21 piles 14 in. by 14 in. by 50 ft.

The whole of the reinforced concrete work for this structure, which comprises the main grain store in connection with the Co-operative Wholesale Society's large flour mill adjoining, was designed by Messrs. Edmond Coignet, Ltd., reinforced concrete engineers, of 125, Gower Street, W.C.1.



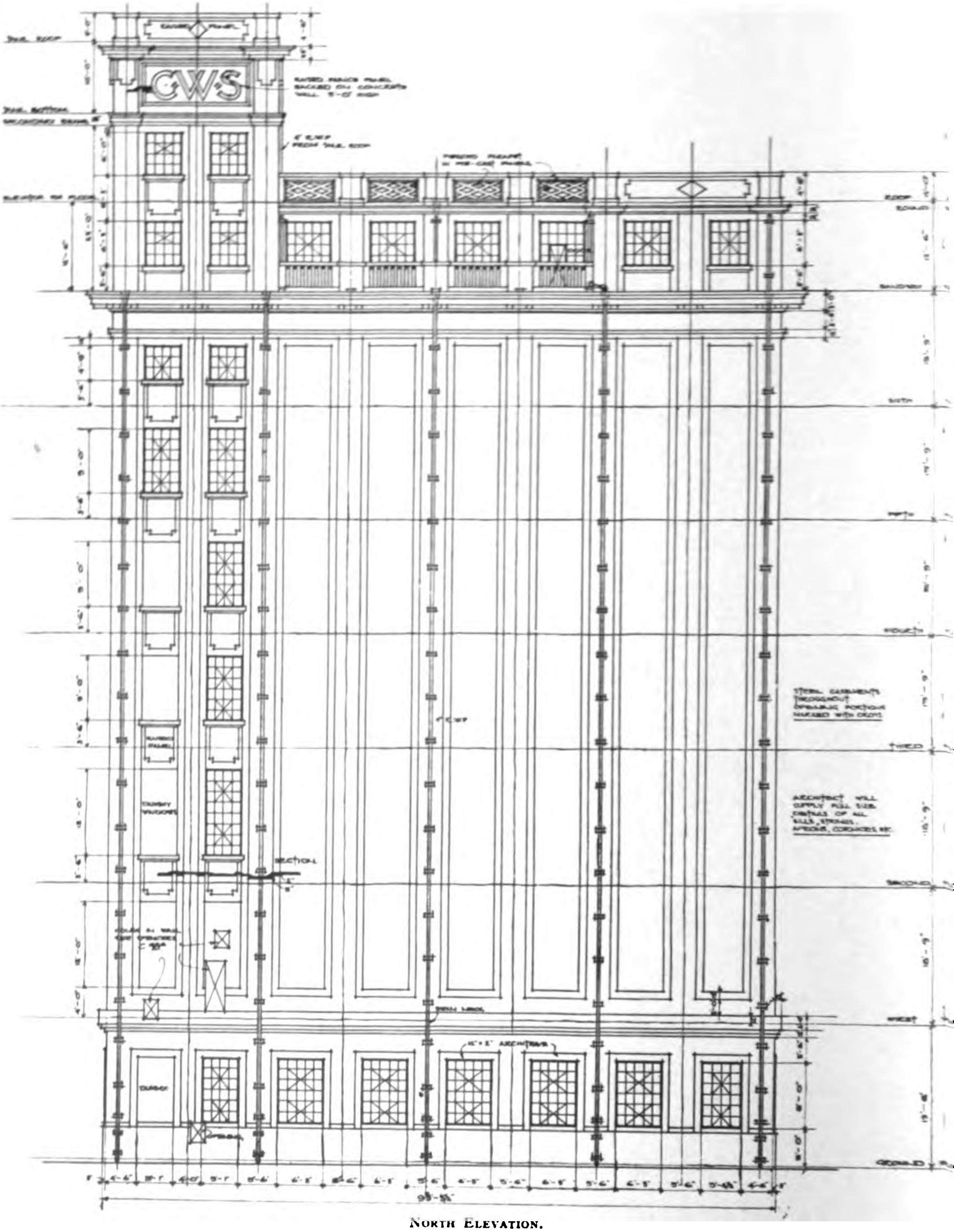
Cross Section.

REINFORCED CONCRETE GRAIN SILOS AT AVONMOUTH. (*See p.* 641.)



[Mr. L. G. Ekins, F.R.I.B.A., Architect.

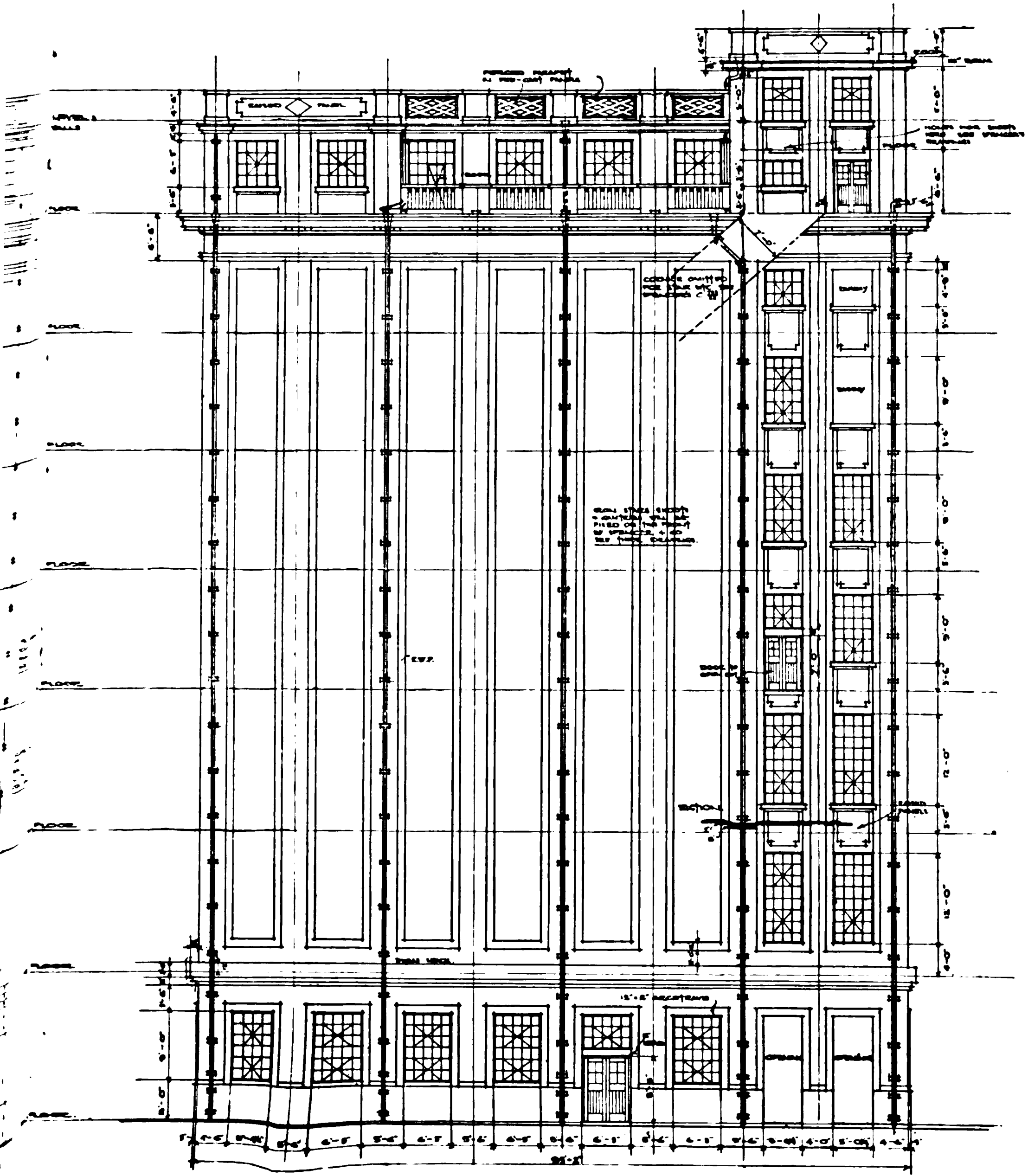
REINFORCED CONCRETE GRAIN SILOS AT AVONMOUTH: DURING CONSTRUCTION. (See p. 641.)


RAISED PANEL
CWS
RAISED ... PANEL BACKED ON CONCRETE WALL
ROOF
BALCONY
SIXTH
FIFTH
FOURTH
THIRD
SECOND
FIRST
GROUND
STEEL CASEMENTS THROUGHOUT OPENING PORTIONS MARKED WITH CROSS
ARCHITECT WILL SUPPLY FULL SIZE DETAILS OF ALL SILLS, STRINGS, APRONS, CORNICES ETC.
RAISED PANEL
DUMMY WINDOWS
SECTION
DUMMY
ARCHITRAVE

NORTH ELEVATION.

REINFORCED CONCRETE GRAIN SILOS AT AVONMOUTH. FO

[The East and West Elevations



SOUTH ELEVATION. [*Mr. L. G. Ekins, F.R.I.B.A., Architect.*

R THE CO-OPERATIVE WHOLESALE SOCIETY, LTD. (*See* p. 641.)

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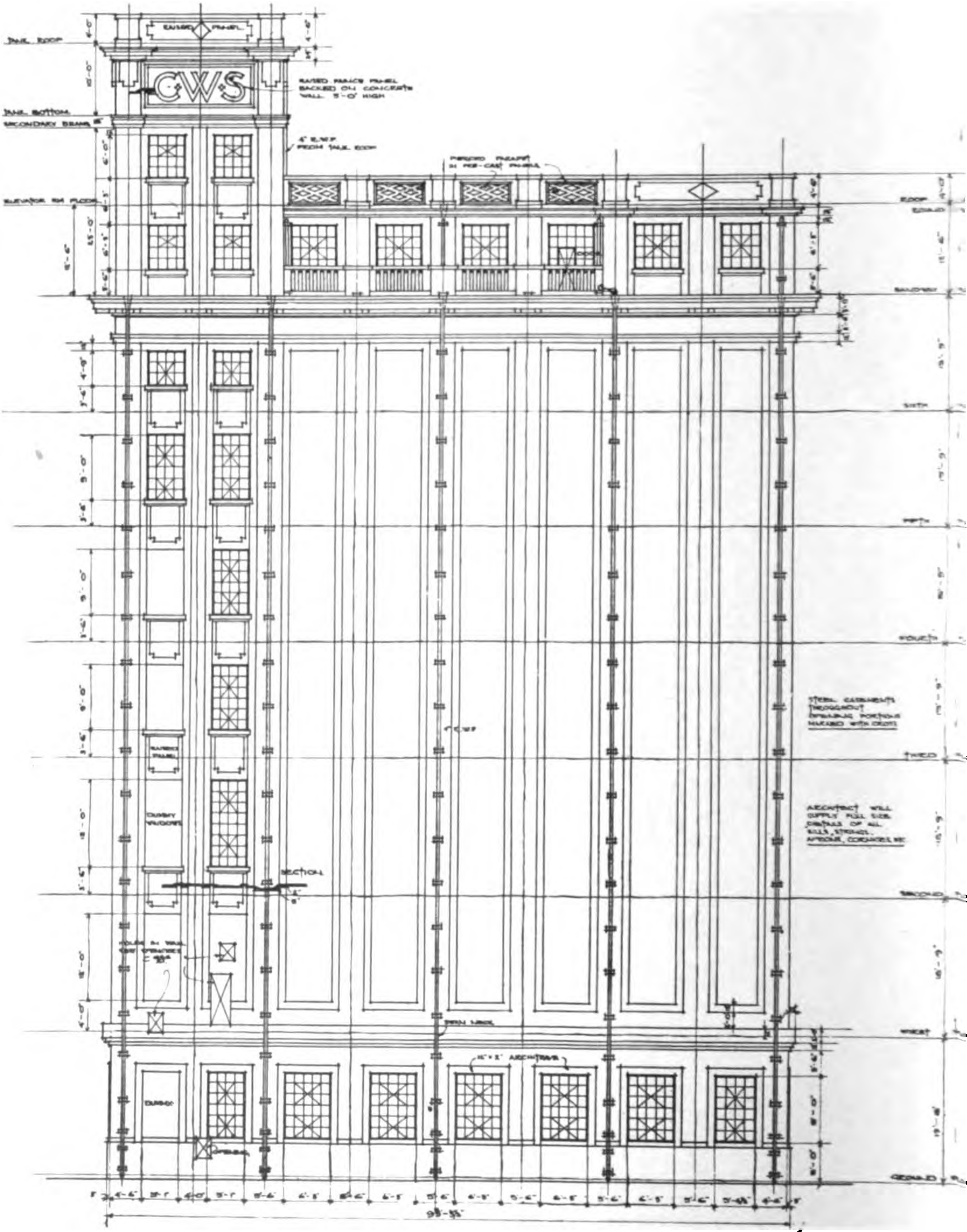
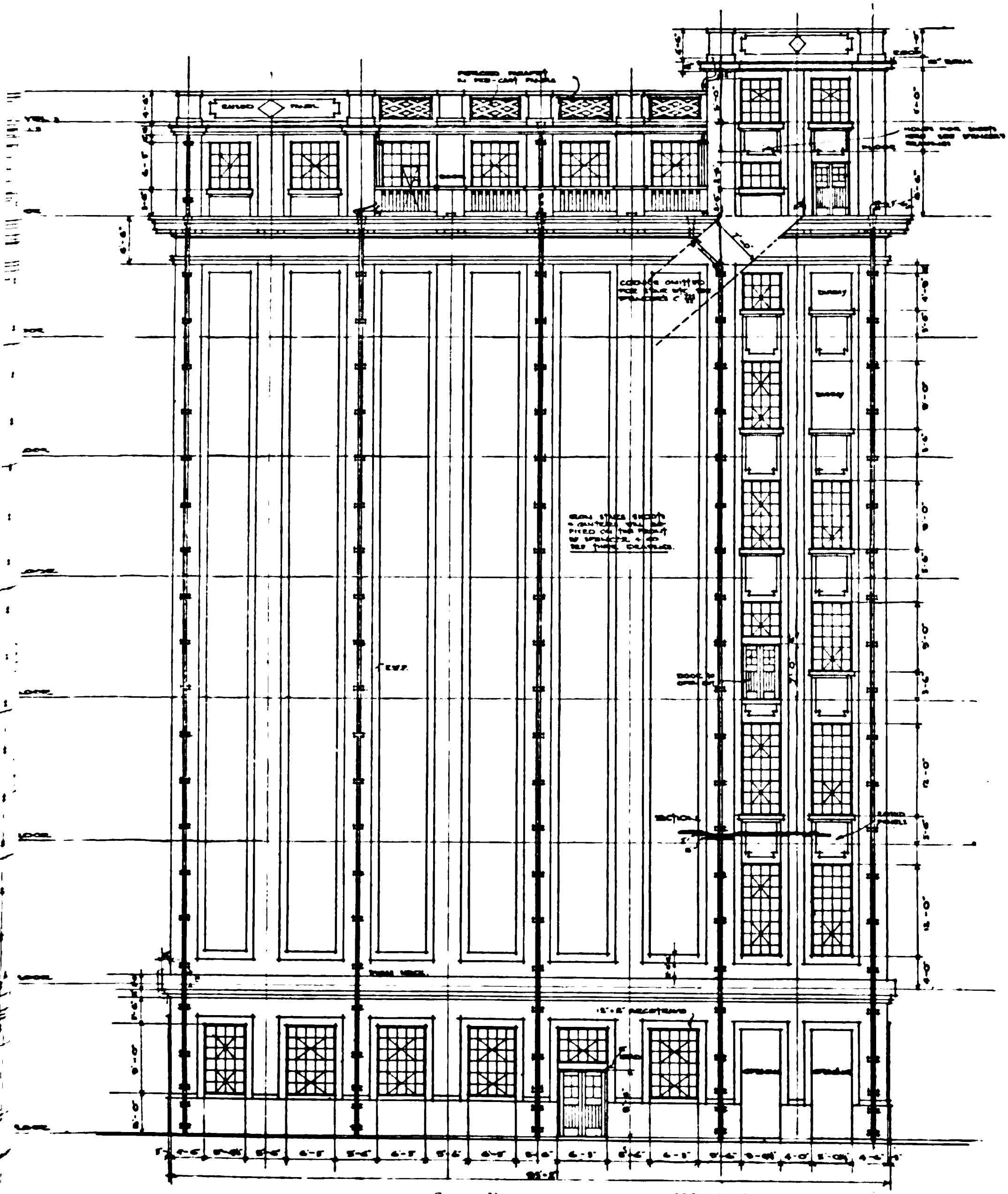

RAISED PANEL
TANK ROOF
CWS
RAISED FAIENCE PANEL BACKED ON CONCRETE WALL 5'-0" HIGH
TANK BOTTOM
SECONDARY BEAMS
4" R.W.P. FROM TANK ROOF
ELEVATOR 9TH FLOOR
PIERCED PARAPET IN PRE-CAST PANELS
ROOF
BALCONY
SIXTH
FIFTH
FOURTH
STEEL CASEMENTS THROUGHOUT OPENING PORTIONS MARKED WITH CROSS
THIRD
ARCHITECT WILL SUPPLY FULL SIZE DETAILS OF ALL SILLS, STRINGS, APRONS, CORNICES, ETC.
SECOND
SECTION
DUMMY WINDOWS
RAISED PANEL
FIRST
DUMMY
ARCHITRAVE
GROUND
93'-5½"

North Elevation.

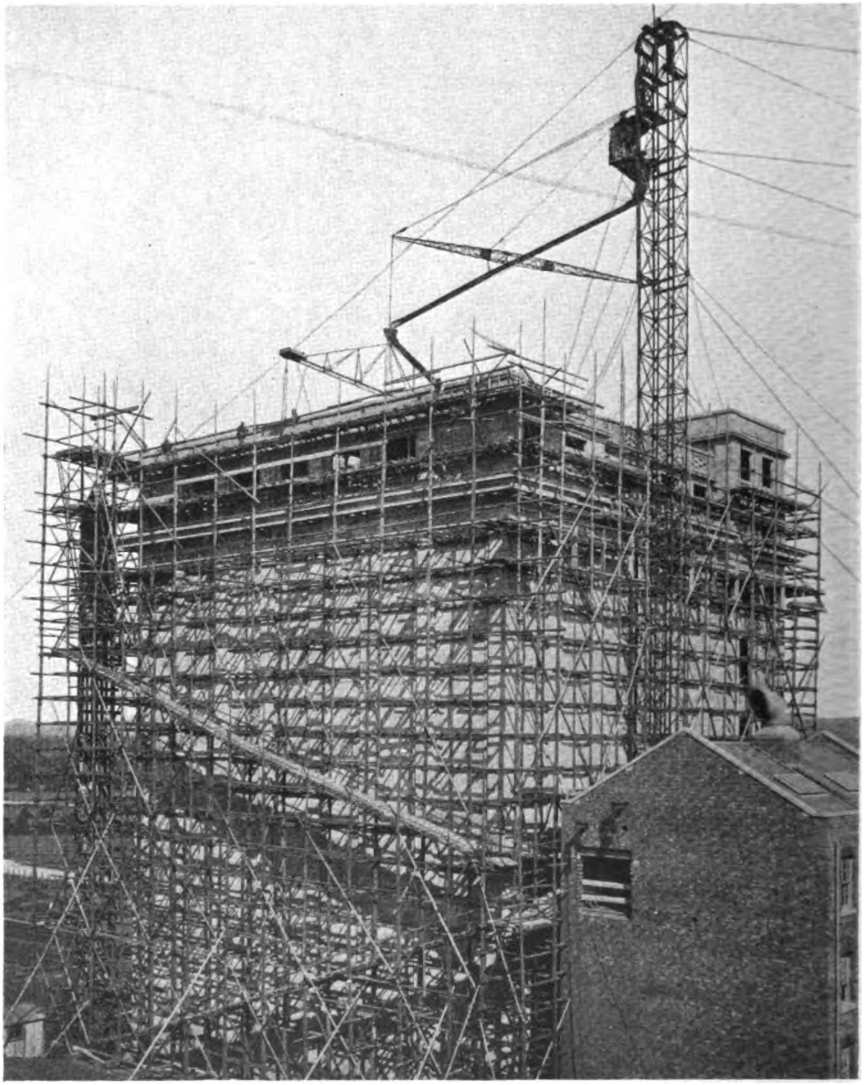
REINFORCED CONCRETE GRAIN SILOS AT AVONMOUTH, FO

[The East and West Elevati



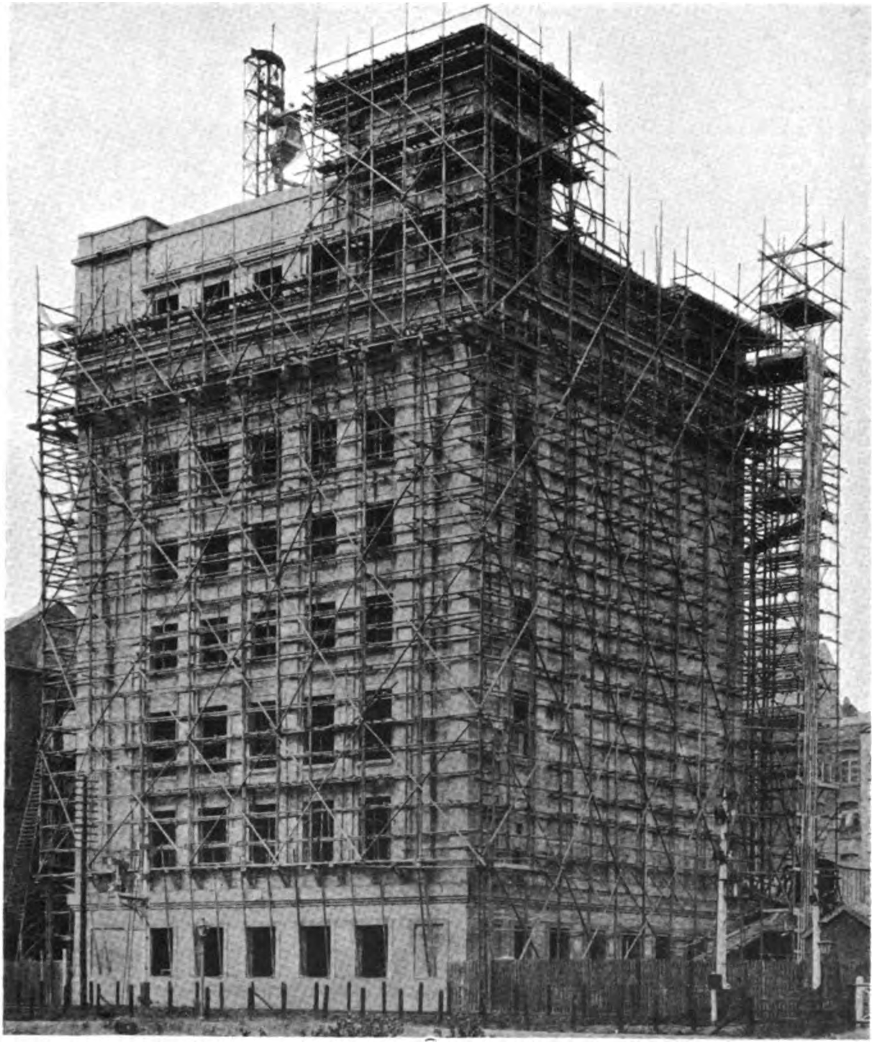
South Elevation. [*Mr. L. G. Ekins, F.R.I.B.A., Architect.*

R THE CO-OPERATIVE WHOLESALE SOCIETY, LTD. (*See* p. 641.)
are given on pp. 678 9.]



[*Mr. L. G. Ekins, F.R.I.B.A., Architect.*

REINFORCED CONCRETE GRAIN SILOS AT AVONMOUTH: DURING CONSTRUCTION. (*See p.* 641.)



[*Mr. L. G. Ekins, F.R.I.B.A., Architect.*

REINFORCED CONCRETE GRAIN SILOS AT AVONMOUTH: DURING CONSTRUCTION. (*See p.* 641.)

TWO REINFORCED CONCRETE GRAIN SILOS IN DENMARK.

WE illustrate herewith two reinforced concrete grain silos recently built in Denmark. The silo at the port of Randers was built in 1919 on the foundation of a storage building that had been destroyed by fire, while that at Nakskov, another Danish port, was erected in 1920.

The Randers silo (*Figs.* 1 and 4) was designed with square bins so as to enable the total load of each bin to be transferred to the columns, these being placed to suit the existing foundation piles. Longitudinally the columns are placed at equal distances, whereas transversely the spacing is varied in order to allow for bins of different sizes, which is an important factor in the economical handling of grain. The silo has 23 bins 49 ft. high, covering an area of 5,000 sq. ft., and has a total capacity of 190,000 bushels.

For the Nakskov silo (*Figs.* 2 and 5) it was decided to use circular bins. *Fig.* 5 shows a longitudinal section. The inside diameter of the cylinders is nearly 12 ft., each bin holding 8,000 cu. ft., while the star-shaped chambers between the circular bins are also used for storage, and each holds 2,500 cu. ft. The plan (*Fig.* 6) also shows how a number of small silos have been made by introducing cross walls in the large bins; in that way altogether 33 bins were made, covering a total area of 5,000 sq. ft. The height of the bins is 45 ft.

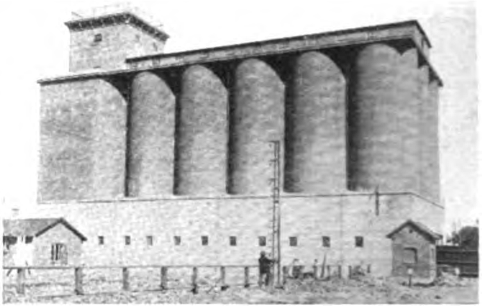


FIG. 2.

The walls—for square as well as for circular bins—are designed to resist a horizontal pressure p, which is expressed

$$p = \frac{\gamma}{m}\left(1 - \frac{1}{e^{mx}}\right) \times tg^2\left(45^\circ - \frac{\varphi}{2}\right) \quad (1)$$

when

$$m = \frac{U}{F} \times tg^2\left(45^\circ - \frac{\varphi}{2}\right) \times tg\varphi_1.$$

U being the circumference of the bin, F the sectional area, x the height to which the silo is filled, γ is the specific weight, and $tg\varphi = tg\varphi_1 = 0{\cdot}65$.

The variation of the pressure (p) will follow a parabola. The vertical pressure (g) may be found from the following formula:

$$g = \frac{\gamma}{m}\left(1 - \frac{1}{e^{mx}}\right) . \quad (2)$$

Consequently, the pressure on the bottom of the bin is derived from formula (2) by making x = the maximum height of the grain, the rest of the weight of the grain being balanced by a vertical friction along the walls.

In addition to bins the two silos each con-

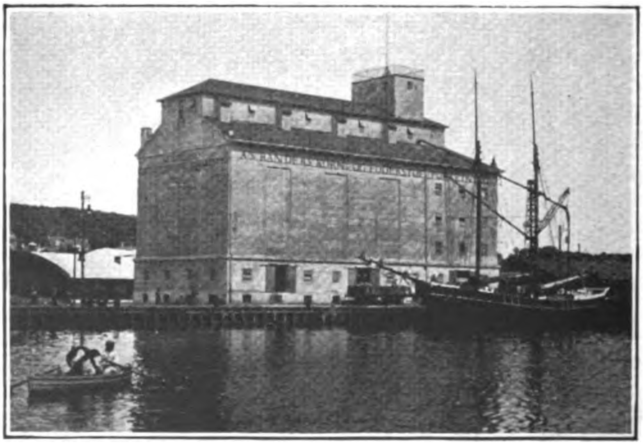


FIG. 1.

tain a store house with plain granary floors (*see Figs.* 4, 5, and 6). The grain can be moved automatically by discharging the contents of the bins on to a belt conveyor running from underneath the bins to the lifts installed in the store house. These lifts carry the grain upwards and discharge it on the top conveyor, which returns it to the bins. In the store house are installed all the mechanical arrangements necessary for the treatment of the grain, such as sifters and mixers, weighers, grinders, etc.

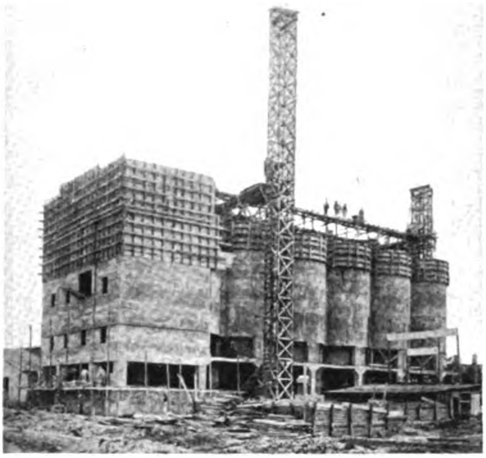


FIG. 3.

At Randers the store covers an area of 2,100 sq. ft. and comprises four floors; the first to fourth floors are alike, while the basement and ground floor run through the whole length of the building (*see Figs.* 4 and 7). The basement contains two belt conveyors and the necessary lifts, a modern drying plant, fed directly from the conveyors, and a general store room.

The Randers and Nakskov silos differ in construction on account of the different requirements. The former is used solely for grain which is to be handled quickly and stored for only a short time, while the latter is used chiefly for storing different kinds of seed grain which requires longer treatment than other kinds of grain.

The Nakskov silo has no basement, the bottom conveyors being placed in a channel in the ground floor, while the lifts are accommodated in a small cellar at one end of the building. The walls and ceilings in the store houses are constructed in reinforced concrete throughout, the flat slab system having been adopted for the ceilings. The photograph of the basement floor of the Randers silo (*Fig.* 7) shows the size of the column bases. It was found necessary to make them of a large size in order to distribute the very heavy load over a sufficient number of the existing foundation piles. In spite of the heavy columns and walls (the outer walls are 3 ft. thick) it will be seen that by this system it has been possible to make a roomy and well-lighted basement out of a restricted area. In the roof is a corridor running through the whole length of the building which contains the top conveyors from which the bins are filled through round openings 10 in. in diameter. The towers are erected on the silos in order that the grain may be elevated, sufficiently high to be discharged on to the belt conveyors.

The roofs, corridors, and towers are constructed in reinforced concrete and presented some difficulty, especially in the case of the Randers silo on account of its roof being sloping, as shown in *Fig.* 7; the lower roofs were built out from the corridor in order to provide access to the exterior row of silos. The concreting of the roof was greatly hampered by the weather, as some of it actually had to be done during frost. On the Randers silo work was started in April, 1919, and the whole of the basement floor was concreted within the first month. In six weeks from the start the columns were finished up to the bottom of the bins (ground floor walls were later on made in brick work). The bottom slabs of the bins were suspended from the walls of the bins by means of very heavy stirrups, and on top of those horizontal slabs conical outlets are made in mass concrete, finished off smoothly on the surface to enable the bins to be completely emptied. The columns are made octagonal in order to occupy the least possible space; the reinforcement consists of circular rings of ½-in. diameter steel bars placed close together.

The exterior walls are 5 in. thick, while the interior walls vary from 4 in. to 5 in., strengthened over the supports. All

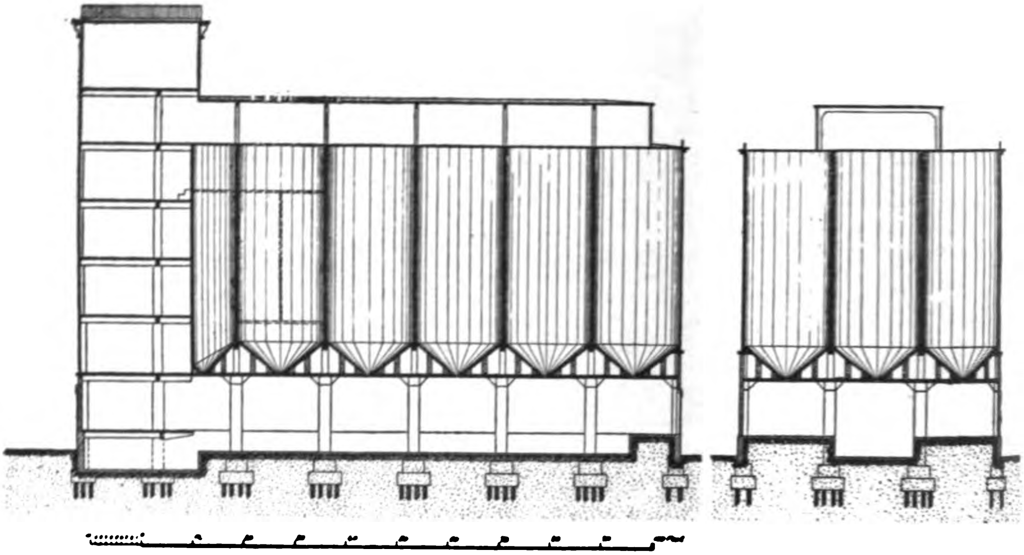


Fig. 5.

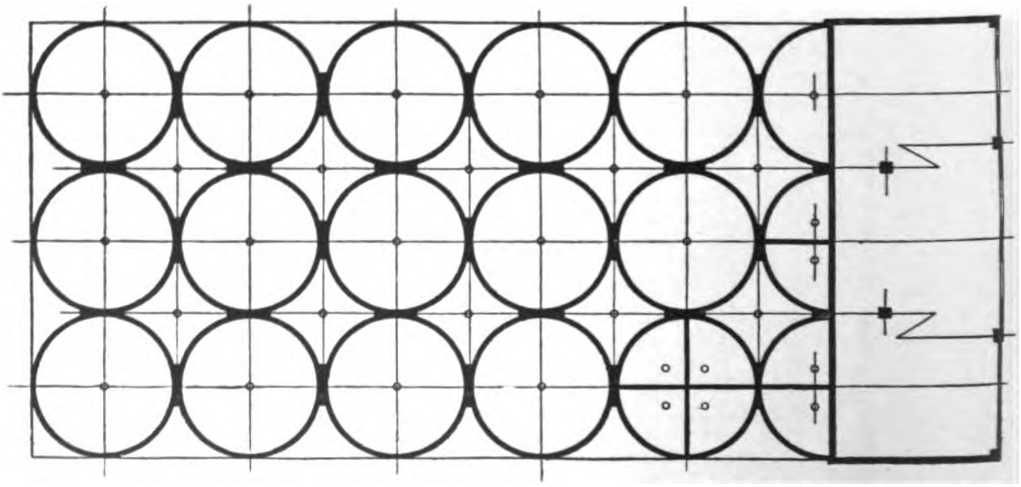


Fig. 6.

REINFORCED CONCRETE GRAIN SILO AT NAKSKOV.

walls have double reinforcement, heavily strengthened at the corners.

For shuttering three sets of moving forms, 3 ft. in height, were used. This method, besides effecting a considerable saving in timber, dispensed altogether with the necessity for scaffolding. *Fig.* 3 shows the Nakskov silo, where the same method was employed, in course of construction; the concrete was poured from the chuting plant and the forms moved upwards as the work proceeded.

As the exterior walls of the silo also form the walls of the outer bins, without any intervening space, great care had to be taken to ensure that they were absolutely impermeable, and this was secured by careful proportioning of the concrete and good workmanship alone; the only after-treatment of the walls was the removal of irregularities and washing twice with cement. That this is satisfactory is shown by the fact that although the walls are only 5 in. thick (at Nakskov 4½in.) they are watertight after four years, and the bins are perfectly dry inside. The concrete consisted of Portand cement, coarse pit sand, and fine pit gravel in the proportion of 1 : 3 : 3. The work was divided up between three gangs for (1) shuttering, (2) placing reinforcement, and (3) concreting, each gang working continuously without being interrupted by one of the others. The concreters worked at night to enable the carpenters to start removing the bottom forms the following morning. The fact that no scaffolding was used made it possible to keep both railway tracks in front of the building in use during the whole construction.

On the interior of the walls of the silos all irregularities were carefully removed, after which one coat of cement grout was applied to fill any pores. On the outside the walls were cement washed twice, and

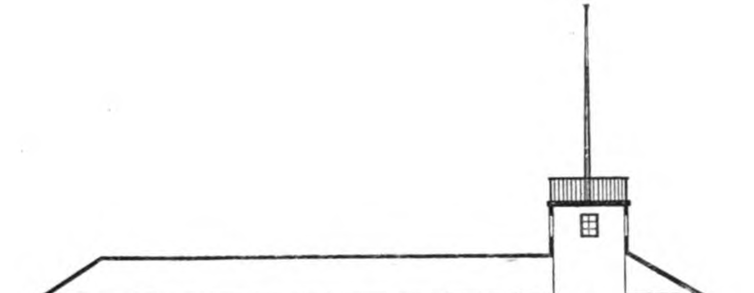


Fig. 4.

this proved sufficient to give the surface a uniform and pleasant appearance.

The Randers silo, in spite of the difficult working conditions, was partly in use only eight months after work was started, and since December, 1919, it has been continually in use. No expansion joints have been provided, and no cracks have appeared.

On the Nakskov silo work was commenced in December, 1919, under conditions somewhat different from those in Randers. The site is reclaimed land, and it was necessary to make pile foundations; about 550 piles of 30 ft. length were driven. The nature of the filling was such that the shuttering for the bottom slabs of the bins could not be directly supported on the mud, and had to be supported on trestles between the piles. The bottom slab was of flat slab construction, and the columns under it were staggered in order to get them beneath the circular bins. The outlets were made of pre-cast segment-shaped reinforced concrete slabs. By this method the work on the outlets did not interfere with the construction of the walls; a considerable reduction in weight was also effected, and it was possible to make passages between the outlets.

The work was carried out in practically the same way as that at Randers, only here the reinforcement and concreting were considerably simplified by making the bin walls circular and with only single reinforcement placed in the middle of the wall. The thickness of the walls is $4\frac{1}{2}$ in. throughout with varying amounts of reinforcement. This work was completed in ten months, part of the building being used for the storage of seeds some time before the whole work was completed.

Both these silos were carried out by Messrs. Saxild & Partners, of Copenhagen and 2, Central Buildings, Westminster, S.W.1.

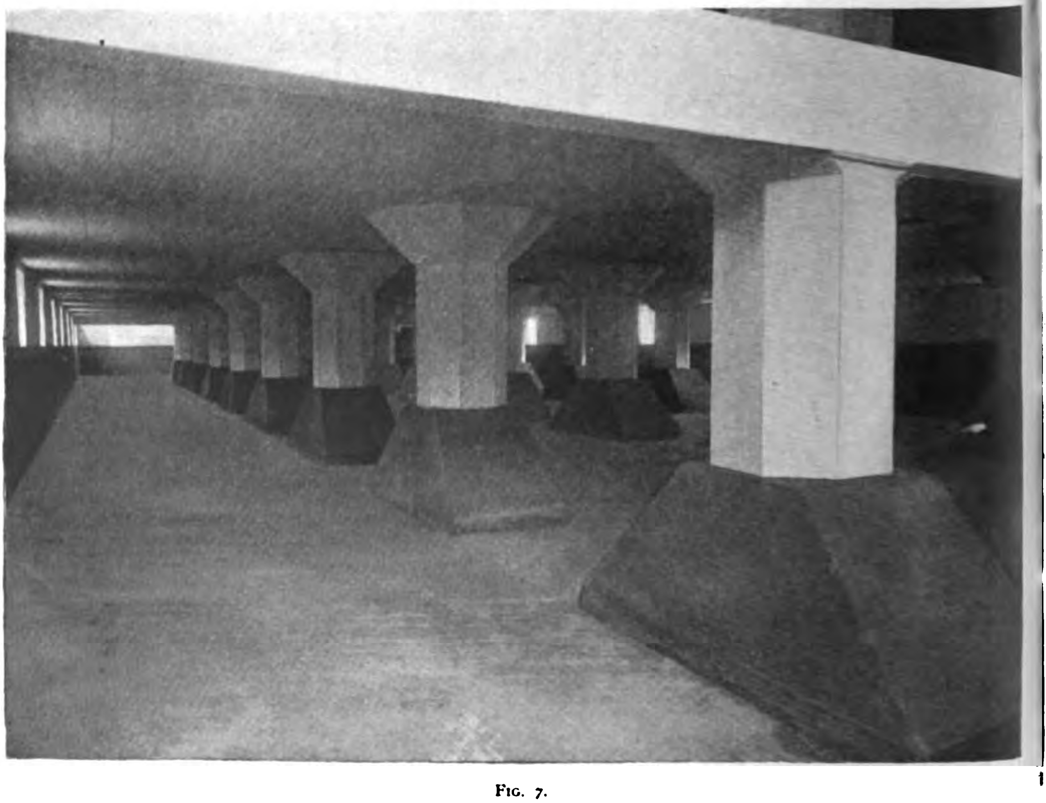


FIG. 7.

POURED CONCRETE HOUSES IN HOLLAND.

DESCRIBING A NEW CONCRETE AND A NEW METHOD OF SHUTTERING.

ALTHOUGH neutral during the war, Holland has not escaped the trade depression and vicissitudes which have affected the belligerent nations, and one of the results, as in practically all countries, is an acute housing shortage. In Holland, as elsewhere, various new methods of construction have been tried, with more or less success. Among the few which have been adopted to any considerable extent is a system of monolithic concrete construction invented by Mr. Greve, a Dutch architect, which possesses two special features. The first is the system of shuttering, whereby the walls may be poured complete or for one story at a time at one operation, and the second is the composition of the concrete of which the buildings are formed.

SHUTTERING.

The shuttering (shown erected in *Fig.* 1) consists of a wooden framework filled

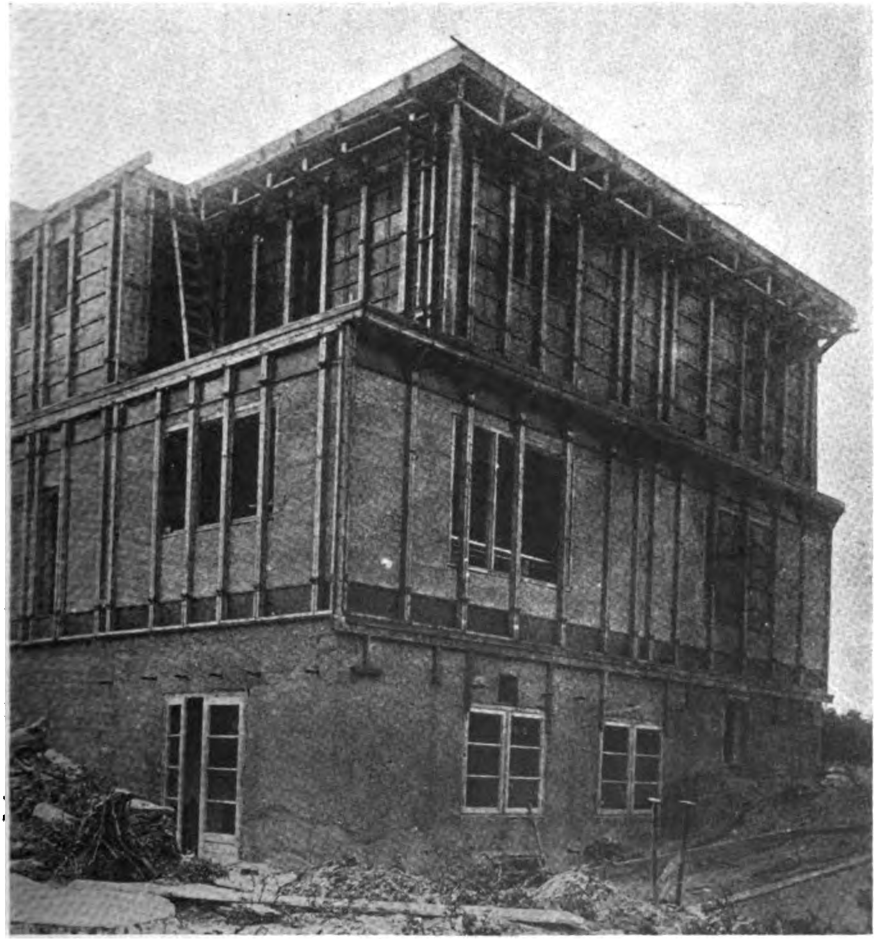


FIG. 1.—SHUTTERING USED FOR THE CONSTRUCTION OF "KORRELBETON" HOUSES: WOOD FRAMEWORK FILLED IN WITH METAL SHEETING.

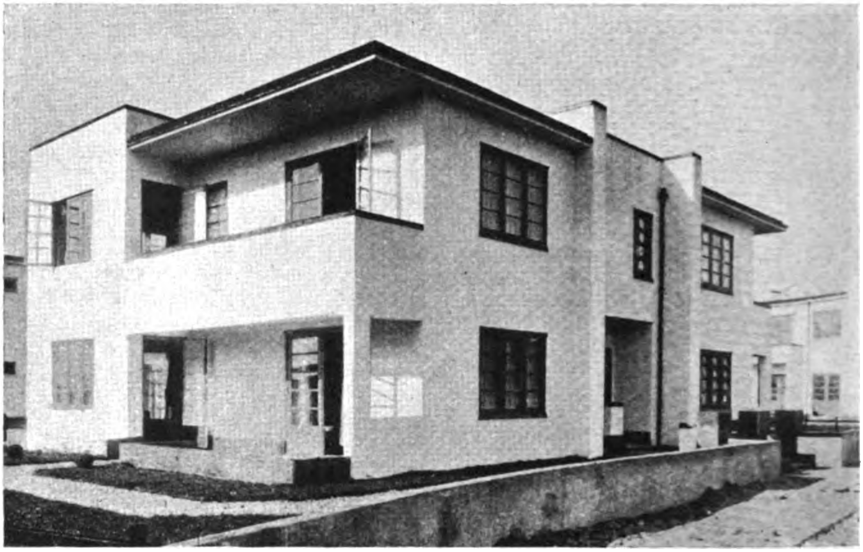


FIG. 3.—PAIR OF HOUSES, SOLD FREEHOLD AT £750 EACH.

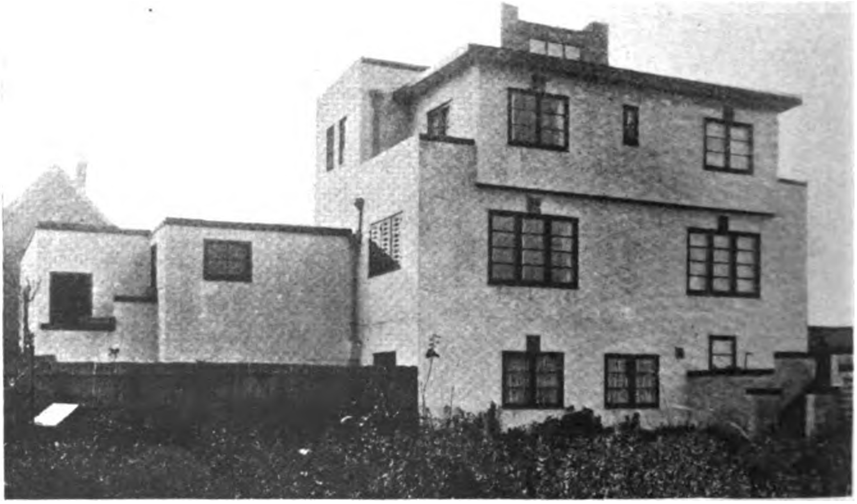


FIG. 4.

HOUSES BUILT OF "KORRELBETON" AT THE HAGUE.

in with metal plates, and is placed in position in sections of a height corresponding to that of each story of the building to be erected. This method has the advantage that the shuttering of each story can be removed irrespective of the shuttering of the other stories, so that where several houses are to be built it is not necessary to wait for the whole of the walling of the first house to become set before any of the shuttering is removed. Thus, while the first house is being completed the foundations of the second house can be prepared, and the shuttering from the ground floor of the first house to be erected, which has hardened in the meantime, is then taken down and used to form the ground floor of the second house. Whilst this is being placed in position, the first-floor shuttering of the first house is being dismantled, and is ready to be erected for the second house by the time the ground-floor shuttering is in place. The same process applies to the second or any number of stories, and a considerable saving in time and formwork is effected. Another feature of the shuttering is that the plates can be removed from the framework whilst it is in position.

"KORRELBETON."

The concrete, known as "Korrelbeton," is made from furnace slag, broken brick, coke breeze, or other angular and porous aggregate to pass a ½-in. mesh, bound together with Portland cement without the addition of fine material to fill the voids. For walls the proportion of cement and aggregate is 1 : 10, while for floors and roofs, in which case a finer aggregate is used, the proportion is 1 : 5 or 1 : 3. The resultant concrete is exceedingly light and porous (see *Fig.* 2), and it is therefore possible to use lighter shuttering than in the case of a concrete made with a heavier and more dense aggregate. The voids in "Korrelbeton" equal from 30 per cent. to 50 per cent. of the total volume of the concrete, and this, it is claimed, renders walls built of this material non-conducting of heat and cold, sound-proof, and fire-proof, while at the same time the light and porous concrete prevents condensation and permits of nails being driven into the walls. Nails, in fact, have a firmer grip in this type of walling than in other types, as when they are driven they find their way around the pieces of aggregate.

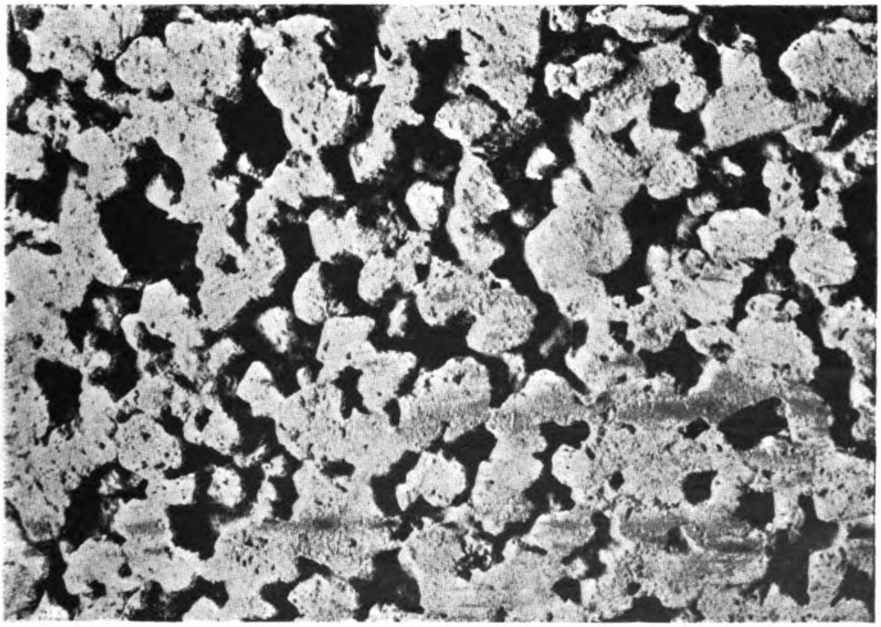


Fig. 2.—Section through Slab of "Korrelbeton": Actual Size.

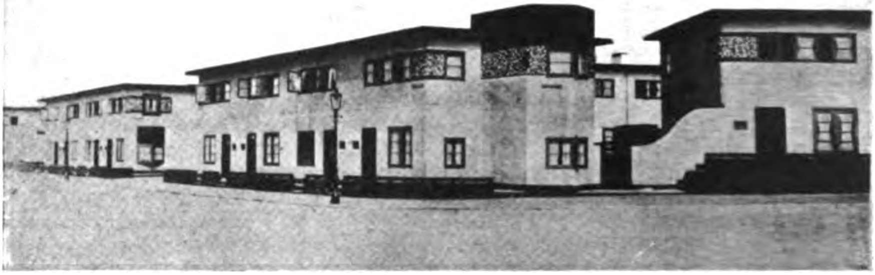


FIG. 5.—BLOCKS OF HOUSES.

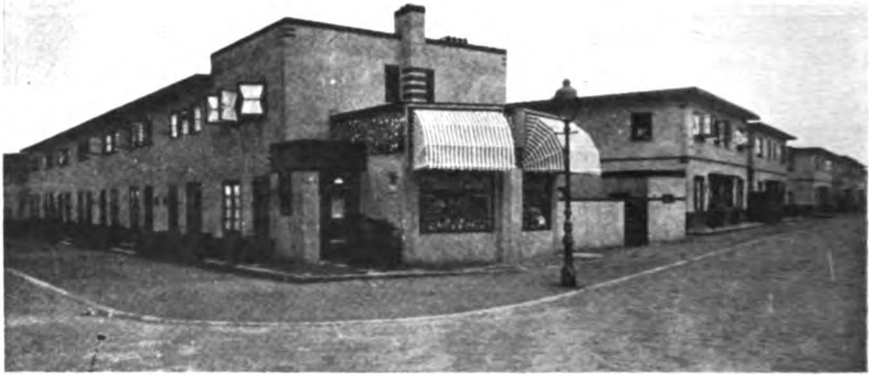


FIG. 6.—HOUSES AND SHOPS.

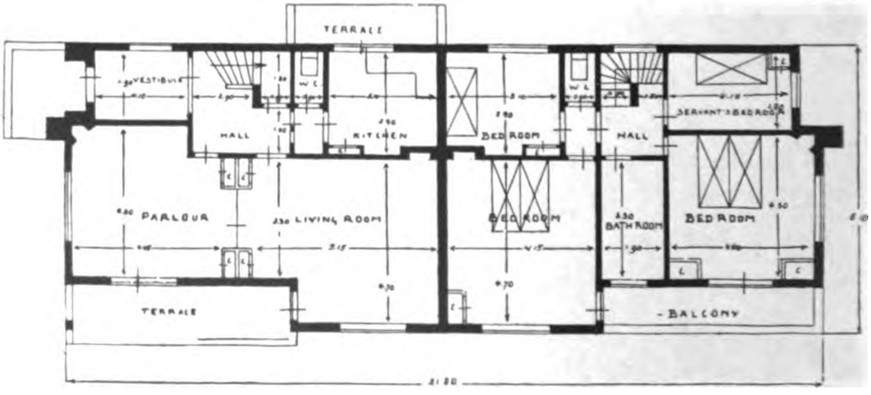


FIG. 7.—PLAN OF HOUSE: THE HOUSES SHOWN ABOVE HAVE THE SAME ACCOMMODATION AS THIS PLAN.

WORKMEN'S HOUSES AT THE HAGUE: BUILT OF "KORRELBETON" AT A COST OF £300 EACH.

METHOD OF CONSTRUCTION.

The walls are rendered weather-proof with a 2 in. coating of cement, and plastered internally. No dampcourses are used, and in this connection we are informed that a piece of walling after standing in water for two weeks showed no signs of dampness above water-level. The outer walls are usually 12 in. thick for foundations and 8½ in. above ground level; divisional walls are usually 4 in. thick.

The method of erecting these houses is as follows. A raft foundation is first laid, consisting of a bottom course of 4 in. of "Korrelbeton" with the same size aggregate as is used for the walls, reinforced with a layer of mesh reinforcement laid 1 in. from the bottom, and a top course of 2 in. of "Korrelbeton" made with a finer aggregate containing only from 10 per cent. to 20 per cent. voids, thus forming a more dense and stronger concrete. The walls are then poured up to first-floor level, and the first floor laid on the same lines as the ground floor. The walls are then carried up to second-floor level and the second floor laid, and so on to any desired height.

It is claimed that the economies which are possible by this system, both in the cost of the concrete and the cost of erection, amount to about 20 per cent. as compared with brickwork. The shuttering is standardised in units which allow any architectural design to be followed, and we are informed that between 300 and 500 houses per year can be built with a plant costing £3,000.

Figs. 3–7 show some houses built at The Hague on this system. The dark-coloured panels to be seen on some of the houses are formed of concrete with the aggregate exposed by brushing the cement from the surface before it has finally set, and this seems to be a fairly common feature in modern Dutch architecture; the concrete for these decorative features is, of course, made of the usual cement, sand, and gravel mixture. The houses shown in *Figs.* 5, 6 and 7 are called "Municipal Labourers' Cottages," and were built at a cost of £300 each, inclusive of site, drainage, electric wiring to points, gas, water, etc. The ample accommodation shown in the plan is made possible by a subsidy from the Municipality to the builder, which allows the rents to be based on a figure below £300; and to satisfy the curiosity of any who may be surprised to find a servant's bedroom in a labourer's cottage we may say that, although ostensibly built for the working classes, these houses are often occupied by tenants with sufficient means to afford a servant.

The Director of the Municipal Building and Housing Council at The Hague, and other Dutch and French building authorities, have reported favourably on this system, which we understand is shortly to be adopted on some further large schemes.

We are informed that a test carried out on 1 sq. yd. of "Korrelbeton" 6 in. thick, such as is used for floors, supported at each of the four corners, showed no deflection when loaded with 2,690 lb. in the centre of the slab.

We have had an independent test carried out in London to ascertain the crushing strength of "Korrelbeton." The sample submitted to us was between ten and eleven months old, and measured 6·66 in. long, with a sectional area of 5·56 in. by 6·5 in., or 37·03 sq. in. The specimen failed at a load of 10·94 tons, that is, 42 tons per square foot, or 661 lb. per square inch.

Mr. C. R. Bartels, of 36, Duke Street, E.C.3, is the London agent for the system, which has been patented in Great Britain and Holland and other European countries.

CONCRETE ROADS.

In view of the importance of the subject, the demand for the second edition of "Concrete Roads" was perhaps to have been expected. The question of building new roads and reconstructing those already in existence to cope with the requirements of the ever-increasing number of fast motor and steam-driven vehicles is one of the most urgent problems confronting Borough Surveyors and the responsible bodies to-day, and it is not too much to say that on its solution the trade and well-being of the country depend to no small extent. It is gratifying, therefore, to know that so much interest is being taken in the concrete road, which has now proved to be the only type of road which will stand modern traffic without a prohibitive initial cost or necessitating the expenditure of excessive sums for upkeep. To quote from the Preface to the book: "Our acquaintance with modern traffic conditions has been comparatively brief, but it has been sufficient to show that the solution of the road problem is not to be found in improving or reconstructing roads on the old lines, but in the construction of a type of road more suited to the weight and speed of the traffic, and in the endeavour to find the ideal road the attention of engineers is now centred on the all-concrete road. So far back as 1906 it was realised in the United States that the new conditions called for something entirely different from the roads which were adequate before the advent of motor traffic, and the fact that 30,000 miles of all concrete roads have been laid in the United States since that date is sufficient evidence that this type of road is not now a subject of experiment, but is accepted as the best, if not the only, road which will stand up to the heavy and fast vehicles of the present day." Previous to 1912 there were, in the United Kingdom, no concrete roads as we understand them to-day, but the rapidity with which they have found favour in this country is shown by the fact that although there were only some 15 such roads here at the end of 1918 there were 281 in June this year.

The chapters include: Concrete Roads in England, Scotland, Wales and Ireland, with latest Official Reports as to their condition, Weight of Traffic, Maintenance Cost, etc.; Concrete Roads in Industrial Works; Concrete Roads in Australia, New Zealand, Canada and America, with latest reports; the Bates Tests of Concrete and other Roads; Mechanical Appliances used in the Construction of Concrete Roads; Concrete Kerbs and Channelling; A suggested form of Specification for Making of Concrete Roads; Pavements, Fence Posts, Telegraph and Lamp Posts, etc.; Useful Hints and Data for making Concrete and the Choice of Aggregates, etc. In the compilation of the book we have had the assistance of engineers who have had a great deal of practical experience in laying concrete roads, and whose work is amongst the most successful in the country.

The book is well illustrated throughout, and the particulars of the methods of construction used by different Borough Surveyors should be particularly valuable to those about to lay new roads—the volume is, indeed, the only publication in which this information has been collected together in a convenient form. The book (price 5*s*. 6*d*. post free), may be obtained from Concrete Publications, Ltd., No. 4, Catherine Street, London, W.C.1, by filling in and posting the form on p. xxvii.

Among the many letters we have received expressing appreciation of the volume, the following from Mr. H. Bryant Newbold, M.S.A., Editor of the *National Builder*, is especially interesting as coming from one who has had personal experience of the Toronto-Hamilton Highway, one of the longest concrete roads in the world, before it was reconstructed. Mr. Newbold writes:—"May I say how interested I am in the second edition of 'Concrete Roads'? The point which particularly appeals to me is the excellent way in which this has been illustrated. The work is almost a gazetteer of concrete roads. In particular, *Figs*. 90, 91, 92, showing the Toronto-Hamilton Highway, appeal to me personally, as along this road, in the condition shown in *Fig*. 90, I, for my sins, once walked. To picture such a road as that shown in *Fig*. 92 could then have been merely the dream of a fevered imagination. To learn that this really exists, from visual proof of a photograph, and to read also of other main roads now constructed in concrete throughout the Dominion, is to become aware that the second step, the first being the railways, has been taken in the development of our Colonies. If this is the effect of concrete, then 'Forward Concrete!' say I."

AN ALIGNMENT CHART FOR THE DESIGN OF REINFORCED CONCRETE SLABS.

By H. M. GIBB.

THE alignment chart on pp. 662–3 has been devised by the writer for the purpose of eliminating entirely the usual calculations necessary to arrive at the thickness and the spacing and diameter of bars in reinforced concrete slabs. The chart is complete and nothing need be calculated.

The chart has been constructed for a beam simply supported and uniformly loaded, i.e.

$$Ms = \frac{wL^2}{8}$$

but it can be used for continuous and cantilever slabs (*Mc*) by entry on Scale A with a value of *w* multiplied by *Mc/Ms*. If the loading is not uniform, after calculating the moment in the usual manner the chart can be used from Scale C onwards.

FOR RECTANGULAR BEAMS.—After the load per lineal foot has been reduced to the load per sq. ft. the chart can be used, and after the spacing of the bars has been arrived at the total number of bars in the rectangular beam will be equal to the width of the beam divided by the spacing of the bars.

Scales A and *B* are self-explanatory.

Scale C gives on one side the bending moment in ft. lb. and on the other side the minimum total thickness of slab which can be used to give the corresponding bending moment. The total thickness of slab has been used in preference to the thickness to the centre of the steel as it is more convenient, and weights of slabs in lb. per sq. ft. are also given in brackets.

Scale D. The depth of concrete below the centre of the steel is given on Scale D, and has been assumed as varying logarithmically from ¾ in. for a 3-in. slab to 1½ in. for a 12-in. slab, instead of varying in steps of ¼ in. as usually given in tables. This does not involve any error of practical importance, and it is as close to the probable accuracy attainable in placing the steel. Scale D also contains the scale of the depth of slab actually used, and its value should never be less than the minimum already read on Scale C.

Scale E gives the area of steel per foot width of slab and the corresponding weight of steel in lb. per sq. ft. of slab.

Scale F gives the diameter of square and round bars.

Scale G gives the required spacing of the bars in inches.

To use the chart apply a straight-edge to the scales A.B.C from the reading on C to C.D.E and from the reading on E to E.F.G.

EXAMPLE.—Design a fully continuous slab

$$\left(M = \frac{wl^2}{12}\right)$$

to carry a load of 200 lb. per sq. ft. on a 9½ ft. span.

A preliminary inspection of Scales A.B.C shows that a 6-in. slab will be necessary.

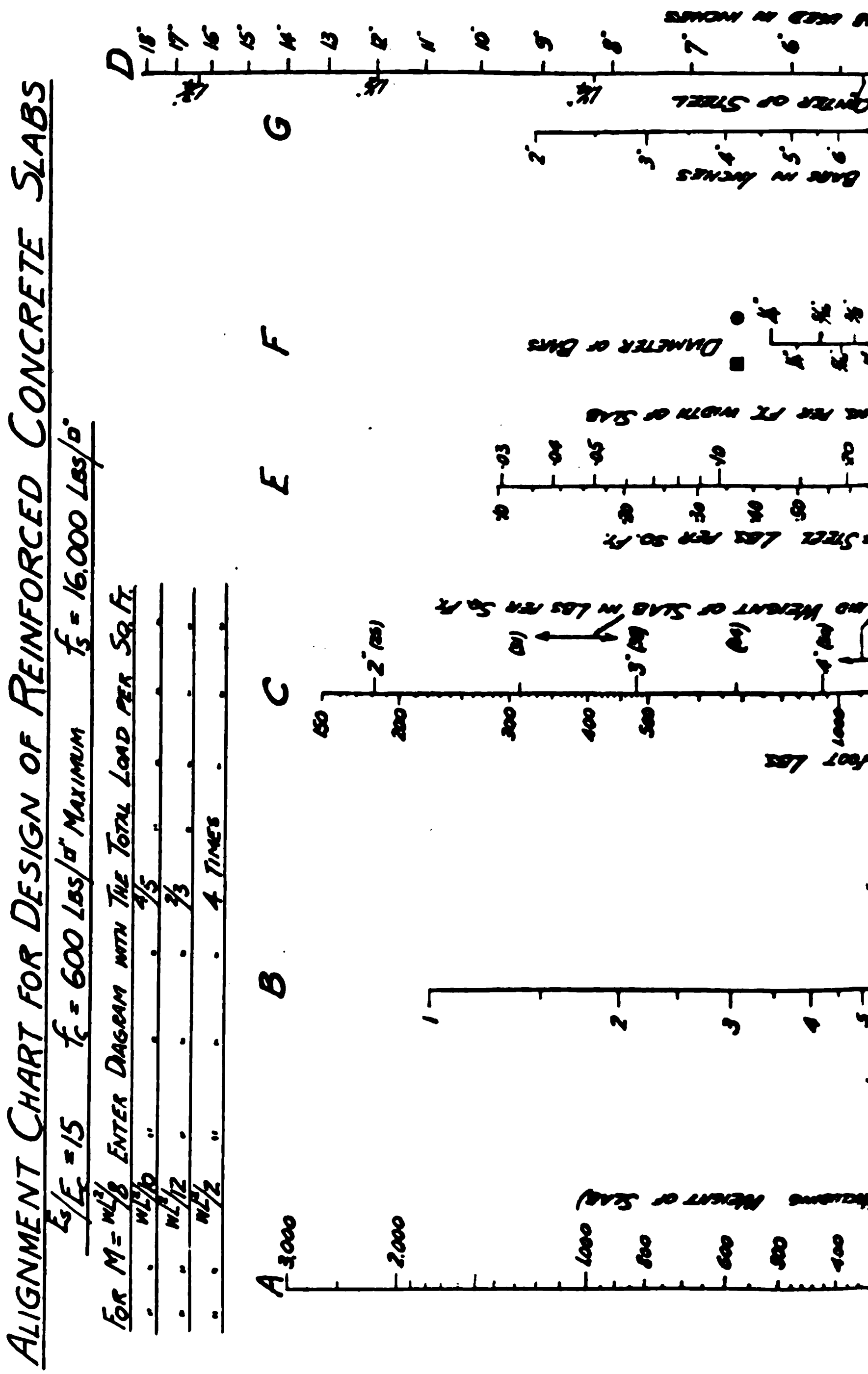
Hence enter diagram Scale A with $w = \frac{2}{3}\,(200 + 75) = 183$ lb. per sq. ft. and align through 9½ ft. on Scale B.

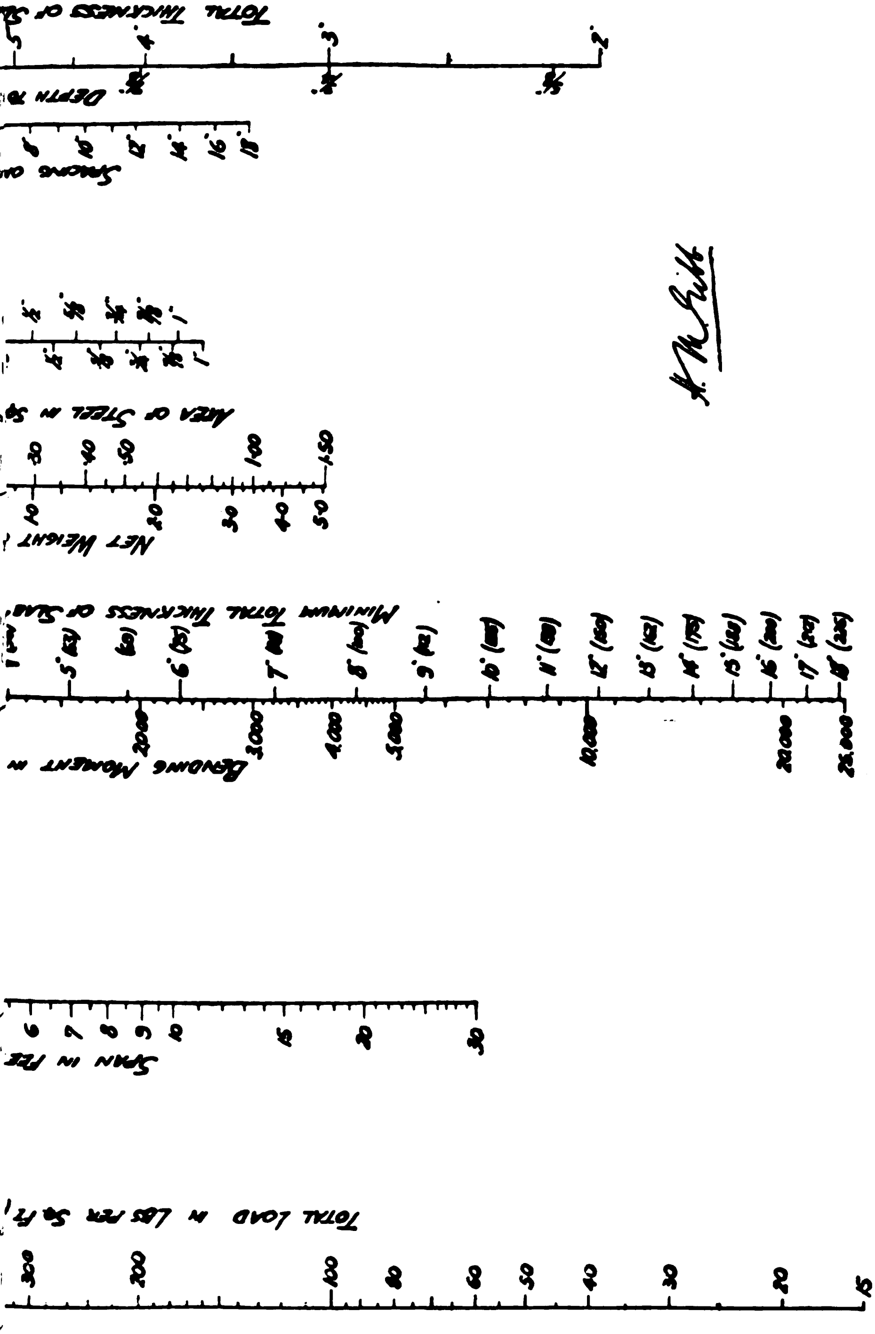
On Scale C this gives a 5·75 in. slab as a minimum. From this point on C align to 6 in. on Scale D, which is the thickness of the slab used, and Scale E reads 0·36 sq. in. of steel per ft. width of slab. From this point on Scale E align through ½-in. round bar on Scale F and read 6¼ in. spacing on Scale G, or 8 in. spacing for ½-in. square bars.

If a greater depth of slab is used than the minimum, Scales C.D.E automatically reduce the area of steel consequent on the increase in depth of slab, as in the above example.

The steel stress remains constant at 16,000 lb. per sq. in., but the concrete stress in this case will be less than the 600 lb. per sq. in. for which the chart is constructed.

The writer has found this chart invaluable for the quick design of slabs and beams. Its advantage over tables is that no interpolation is required, and it is also more complete than any alignment chart so far published.


ALIGNMENT CHART FOR DESIGN OF REINFORCED CONCRETE SLABS
Es/Ec = 15
fc = 600 LBS/□" MAXIMUM
fs = 16,000 LBS/□"
FOR M = WL²/8 ENTER DIAGRAM WITH THE TOTAL LOAD PER SQ. FT.
" " WL²/10 " " " " " " 4/5
" " WL²/12 " " " " " " 2/3
" " WL²/2 " " " " " " 4 TIMES
A
B
C
D
E
F
G
3,000
2,000
1,000
800
600
500
400
1
2
3
4
150
200
300
400
500
1,000
2"
3"
4"
.03
.04
.05
.10
.20
.10
.20
.30
.40
.50
DIAMETER OF BARS
18"
17"
16"
15"
14"
13"
12"
11"
10"
9"
8"
7"
6"
2"
3"
4"
5"
6"
WEIGHT OF SLAB)
FOOT LBS
WEIGHT OF SLAB IN LBS PER SQ. FT.
STEEL LBS PER SQ. FT.
PER FT. WIDTH OF SLAB
BARS IN INCHES
CENTER OF STEEL
USED IN INCHES


Total Thickness of Sla
Depth to
Spacing on
Area of Steel in Sq.
Net Weight
Minimum Total Thickness of Slab
Bending Moment in
Span in Fee
Total Load in Lbs per Sq Ft.

(SEE page 661.)

CONCRETE BUNGALOWS.

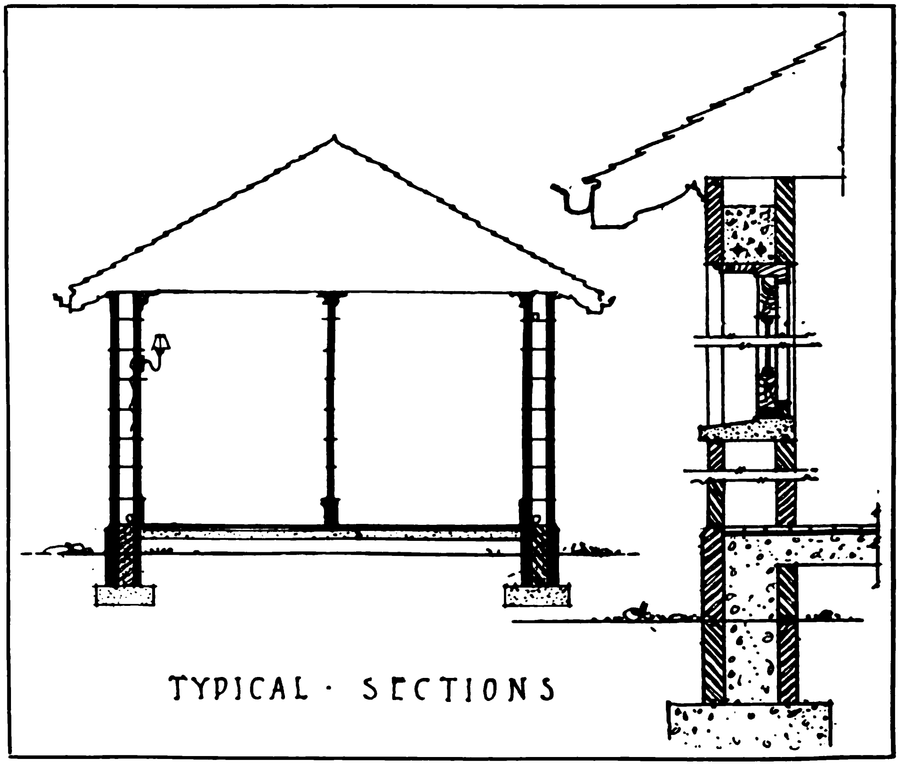
(*Contributed.*)

One of the most fascinating uses to which concrete may be put is in the construction of bungalows of all kinds. The bungalow idea has come to stay, especially at the seaside where so many of us make our way at the end of the week.

The construction throughout these buildings should be studied from the economic viewpoint, as a holiday bungalow may be in occupation only a small part of the year; and its initial cost should not, therefore, be a financial burden to the owner. The planning should, of course, aim at reducing all unnecessary labour. The owner should be able to run the place himself or herself, without any servant; and the furniture should be simple and strong, with plenty of cupboards and drawers, so that everything may be put away after each visit till required again.

The main idea of the plans suggested is that there should be one large open living-room, all other parts of the bungalow being subsidiary thereto. The way of these structures should invariably be built on the hollow or cavity principle, so as to keep the building perfectly damp-proof in rainy weather; also, the hollow spaces may be used for forming chimney-flues and holding service pipes, wiring, etc. For this purpose thin slabs or tiles are very effective, both from the constructional and artistic standpoint. The dimensions of the slabs may, of course, vary with the architect's requirements, but usually a height of 12 in. gives a very good stone-like effect. The aggregate for this slab concrete may be gravel, with the larger pieces taken out, or crushed stone, the larger pieces being not more than one-quarter of an inch cube. Breeze is not a suitable material, as it is not possible to obtain with it the fine surface effects of cement sand.

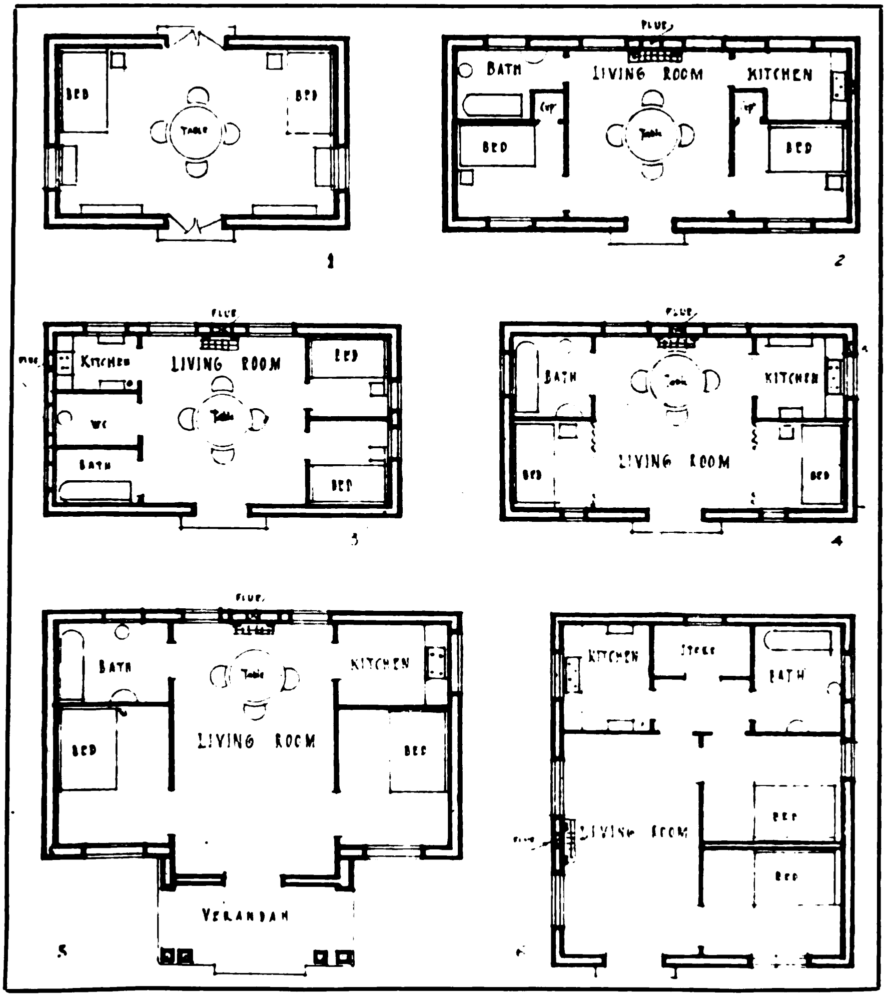
The thickness of the slabs may be re-



duced down to 2 in. in order to lessen the amount of the material used per foot super and so cut down the cost, and also to lighten the task of handling. Further, these slabs are easily cut by being sawn down to about a quarter of an inch and the part not required broken off on the edge of a table or bench. This cannot be so easily accomplished with slabs over two inches in thickness, or with the ordinary blocks, for which numerous "specials" are usually required. The cost and time of making special moulds is thus obviated and all the slabs can be cast on the job to avoid transportation.

To negotiate angles and to form door and window reveals an angle-piece is required of the same thickness as the flat slab or tile. This angle should be finely finished on both sides as it will be required for outside and inside work, and when reversed will break joint. There should be no connection between the outer and inner slabs other than narrow metal ties, along which there will be no creeping of moisture. Complete insulation against moisture, heat, and cold can only be obtained by cavities which are continuous both horizontally and vertically. The sectional sketch illustrates the typical use of these cavities, and the general method of holding and separating the slabs.

The interior partitions may be built



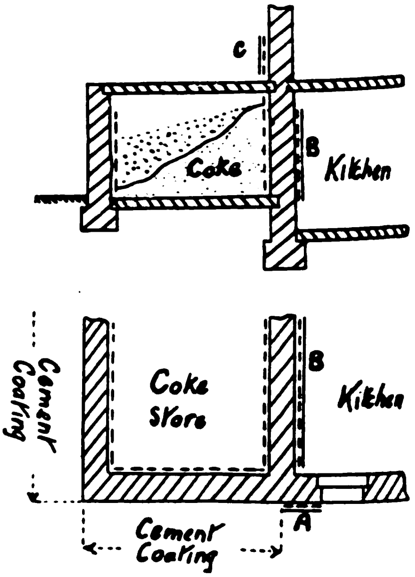
up in single 2 in. slabs or in double formation, with an intervening cavity where it is required to be sound-proof. The metal ties should in the latter case be omitted entirely or used sparingly, as it is only the disconnected continuous cavity that gives it that quality.

If the slabs are finely made both as regards material and workmanship, and set up by skilled labour, their surfaces and angles are so accurate that a wall of perfect smoothness may be obtained on which may be applied wall paper direct, or the joints may be puttied up and the surface treated with paint or attractively pointed and then tinted to taste. 'Skirtings, picture-rails, and cornices may be cast in with the slabs at one and the same time, and so save the extra cost and labour of plaster work. The work of the plasterer is, indeed, confined to the finishing of the ceiling only.

Concrete bungalows thus constructed are very cheap (cheaper than ordinary brickwork), and are quickly built; such cottages as the larger of those illustrated in the plans may, in fact, be built for about £200. From the æsthetic standpoint they are admirable, as the surface textures and colour effects obtained by using the rodding process and light colour cements are both varied and beautiful. The thickness of the wall is regulated by the length of metal ties, the thicker walls being no more costly than the thinner ones. The former are to be preferred on account of the protection they give to the doors and windows, and the shadow effects are very pleasing.

DAMAGE TO CONCRETE BY COKE.

Scarcely enough attention is paid to the risk of concrete being damaged by coke, states *Beton u. Eisen*, especially when the latter contains sulphur derived from imperfectly oxidized pyrites in the original coal from which the coke was made. It is by no means unusual for a coke to contain 1·2 per cent. of sulphur, and it is not surprising if such a material, when wet, attacks plaster, mortar or even concrete. In one instance an objectionable white scum (erroneously regarded as " wall nitre ") occurred at three places, A, B and C, in a house (*see* diagram), and, on examination, was found to be due to wet coke being stored as shown. The moisture, being unable to escape owing to lack of ventilation, was absorbed by the walls, carrying with it sulphur from the coke in solution and depositing the resultant salts on the walls at the parts where evaporation was most rapid. On replacing part of the mortar-finish by a cement-sand mortar, which set to a hard and very dense mass, it was found that the scum did not form on the parts so protected, whilst it continued to increase in those parts where a more porous material was used.



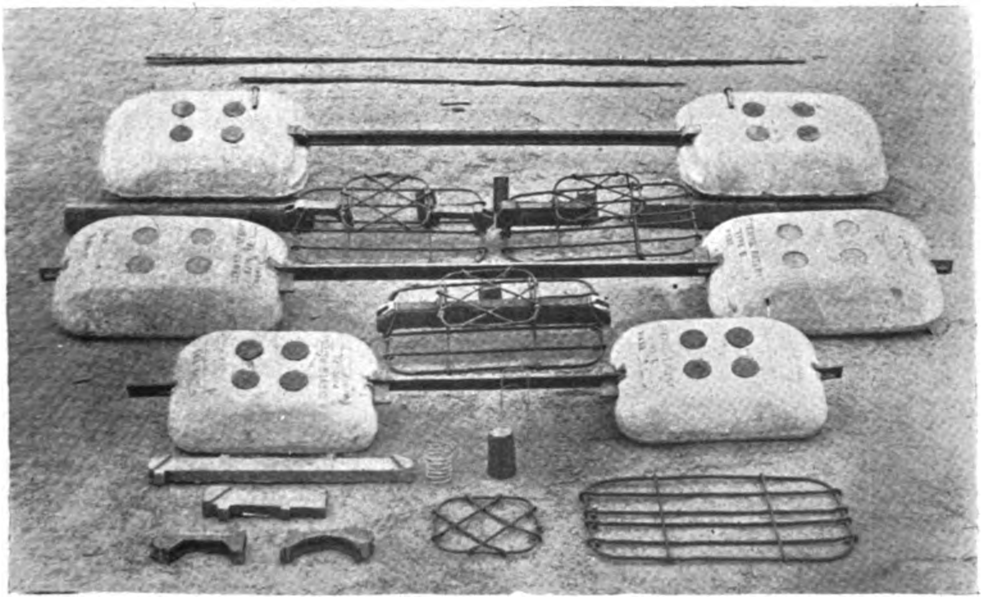
CONCRETE RAILWAY SLEEPERS IN INDIA.

CONCRETE railway sleepers of various patterns have been tried in different countries during the past twenty years, some of which have been described in this journal. Among these is the "Stent" sleeper, which is being extensively used in India. This sleeper has passed the experimental stage, and has been definitely accepted as satisfactory by the Indian Government Railways. It is now manufactured on mass-production principles, and in addition to the economic interest from a railway's point of view there is much in the manufacturing processes that will be of interest to users and producers of pre-cast concrete units.

The sleeper is the invention of Mr. D. H. Stent, who until recently was a divisional engineer on the Bombay, Baroda & Central India Railway, where he spent some ten years experimenting with and perfecting his design. The Indian and Far Eastern rights of Mr. Stent's patents were acquired in 1919 by Messrs. Bird & Co., of London, Calcutta, Delhi, etc., who built a factory for their manufacture at Delhi in 1920. So far, we are informed, the manufacturers have not been able to increase production fast enough to keep up with the demand.

The sleeper is of the three-piece, or tie-bar type where, as shown in the accompanying photograph, two concrete blocks are joined together by a metal tie-bar in the same manner as is the practice with cast-iron sleepers. India has always favoured the permanent sleeper, where the cost has been economically practicable, and the forms most extensively used have been of the cast-iron "plate and tie-bar" type. The chief features of the "Stent" sleeper, apart from the ease of portability obtained by the adoption of the three-piece principle, are the method of fastening the rails and the improved form of tie-bar and fittings used. The latter allows of considerable economy in cost, and at the same time greater efficiency is claimed.

The rail fastening is by screw, dog, or round spike, according to the type of rail or chair used, driven into compressed-wood plugs set in the body of the concrete blocks exactly as such spikes are used with timber sleepers. The plugs are of sufficient size to allow of considerable latitude in the positioning of the spike and to enable various sizes of rails or chairs to be laid on the one standard sleeper without altering the length of



DIFFERENT TYPES AND SIZES OF "STENT" CONCRETE SLEEPERS, WITH THE REINFORCEMENTS AND PARTS.

tie-bar or any fittings, an advantage so far unobtainable with any permanent sleeper. The inadaptability of the iron sleeper to change in rail sections, as in renewals with a heavier rail, has always militated to some extent against the more extensive use of permanent sleepers in India, and it is presumably largely due to this feature that they have not been more generally adopted in western countries.

The works at Delhi cover some 8 acres and are now entirely given up to sleeper manufacture. The present output is at the rate of about 200,000 sleepers per year, but the works have been laid out with the view of ultimately obtaining from 400,000 to 500,000, and plant extensions and improvements are being gradually carried out to effect this.

There are several new and interesting departures in the process of manufacture, and Messrs. Bird & Co. are to be congratulated on the high standard of efficiency obtained. Of the various processes the fabrication of reinforcements, the stone-crushing plant, the concrete mixing and feeding plant, and the general treatment of the concrete, by which the high standard of finished product is obtained, will chiefly interest concrete users not otherwise interested in railway questions. Another interesting process is that of the timber shop where the plugs are made, treated and driven. Timber treatment does not ordinarily enter into concrete questions, but it would appear that there is scope for extending the use of the Stent plug as a means of securing heavy bodies to concrete walls and floors. In concrete factory buildings the latitude in positioning that these plugs give should be of considerable value for bolting machines, shafting brackets, etc.

REINFORCEMENT.

The reinforcement consists of closed frames of ordinary light round bars of from ¼-in. to ⅜-in. diameter, termed "grids." The lower grid takes the tensile stresses, and upon this chiefly depends the strength of the sleeper and the axle load it will stand. The sleepers are made for broad, standard, and metre gauges, and for axle loads of from 8 to 26 tons. Each grid is a complete rigid unit cross-braced so that none but the roughest handling will distort it. Another essential internal part of the concrete block is the tubular cast-iron guide which receives the end of the tie-bar and in conjunction with which the tie-bar fastenings are arranged, thus providing metal to metal surfaces for all moving parts and preserving the concrete from abrasion. This guide is assembled with the reinforcement and forms a distance-piece between the upper and lower grids correlating the whole. The reinforcement is held in the mould by the guide, which is fixed in the mould in the same manner as it is later fixed to the tie-bar, thus ensuring correct positioning and gauging and that no reinforcement shall get out of place during the process of concreting.

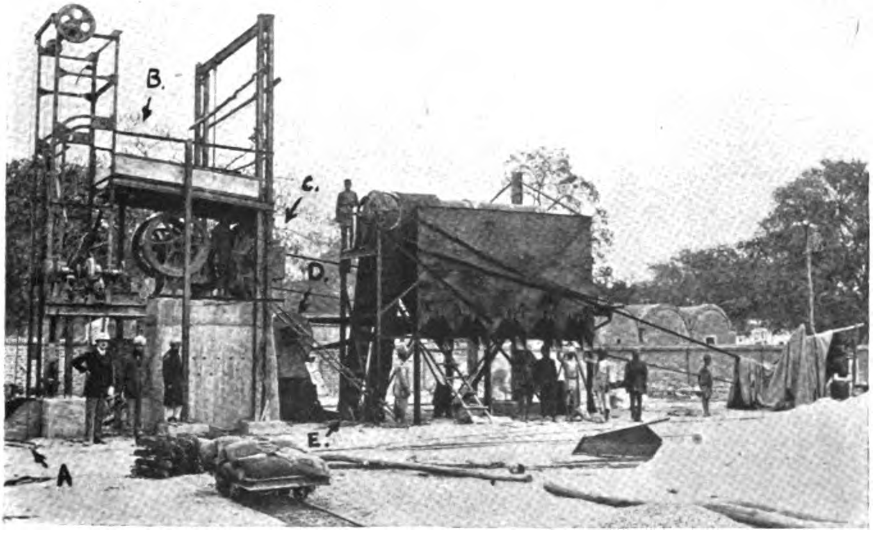
The bars for the grids are straightened and cut to length in an automatic machine, but no bar is pre-bent. The grids are made up on former plates from the straight-cut bars, using a bending tool for locking over the ends, and the grid is removed from the former plate, locked rigid and complete ready for assembling with the cast-iron guide. This method of hand-forming and assembling is that devised by Mr. Stent during earlier experiments on the B.B. & C.I. Railway, and so far no machine has been found to improve on it or reduce cost. It is said that the labour cost of production, even with this light steel, is under one shilling per cwt. all-in, including handling the raw material and finished product in and out of stores.

AGGREGATE AND STONE-CRUSHING PLANT.

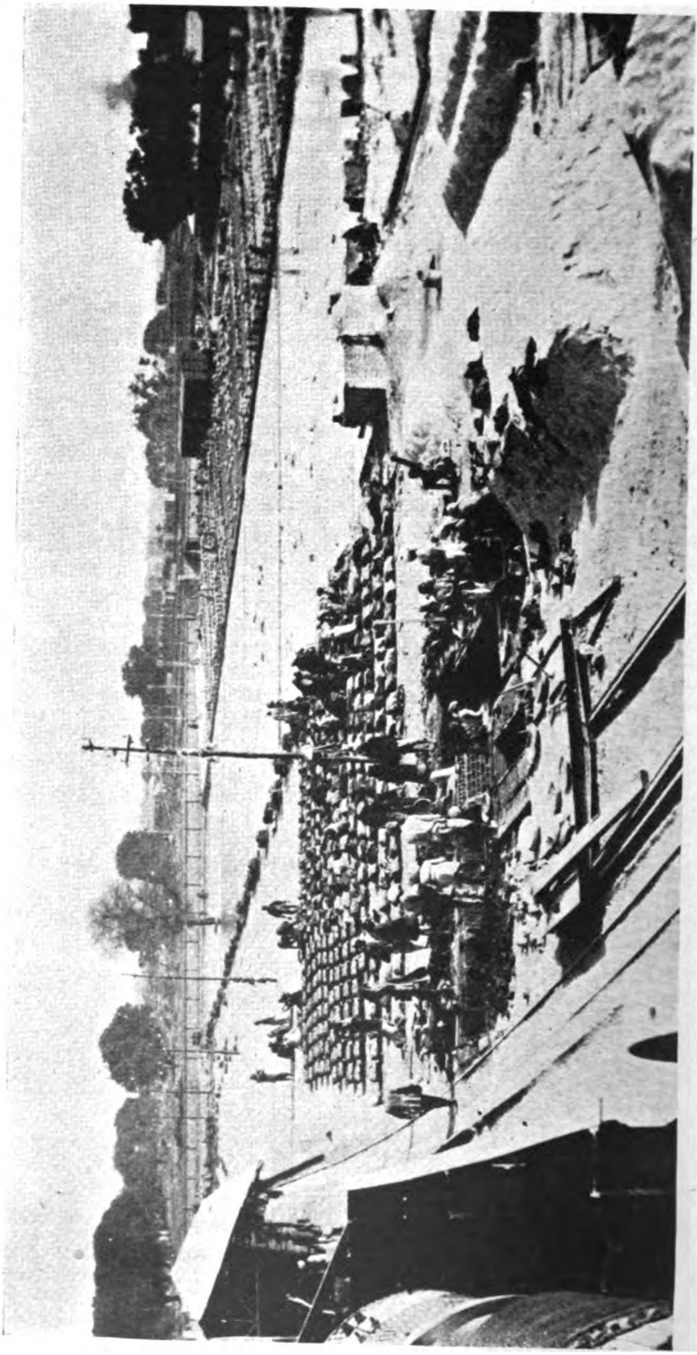
The stone used for the aggregate is a pure silica quartzite, and is extremely hard—a very necessary feature if abrasion of the rail-seat from the continual hammer action of the rail is to be kept at a minimum. Originally the stone plant consisted of the swing-jaw type of crusher only, but they were found uneconomical and unfit for reducing such a hard stone from heavy rubble down to the ¾-in. to ¼-in. sizes required. More recently a new stone-crushing plant (shown in the accompanying illustration before final completion and roofing) has been installed to Mr. Stent's design, and has several features of interest. It is designed to reduce handling to a minimum with the least expenditure in elevating power while giving a steady supply, synchronising with the requirements of the concrete-mixing plant, and workable by inexperienced Indian labour. Though the local conditions may be of a special nature not obtaining elsewhere, the general principles might be applied with advantage to many plants dealing with quarried stone rubble.

The stone rubble is brought from the stone stacking-ground by 1-cu. yd. tip wagons, which discharge into an elevator skip (A). This skip delivers the stone on to a sloping platform (B), from which the feed at the mouth (C) to the stone-crusher below is regulated by hand. The crusher reduces the 9 in. to 15 in. rubble down to 2 in. to 3 in., and the metal then passes to the Hadfield disc crusher at (D), where it is reduced to sizes from ¾-in. maximum down to ⅛-in. and below. The fine crushed material then falls to the Baxter internal bucket elevator at (E), which delivers it to the rotary punched screen by which it is sorted into four sizes and dropped into 2-yd. capacity steel bins. The various crushed sizes are drawn off from the bin chutes into tip wagons as required, and the tip wagons run down some 40 ft. to the concrete plant where they again discharge into the mixer elevators. The stone plant was designed to produce 100 yds. per day, but it has been found that double this production can be obtained and it would be greater still with a softer stone. The saving in power and labour with this plant as against the old has proved even greater than was expected, and the direct costs of production are believed to have been reduced to under one-third of the earlier costs.

A feature of the plant is that all fine dust which would pass through a 30-mesh sieve is drawn off by a fan from the down chute before it enters the elevator drum at E, and is carried to a dust flume to settle. There is possibly more dust with this hard stone than might be the case with softer stones, but in any case removal of the dust in this manner gives a better aggregate and makes for better conditions for the labour working around the plant. A good washing system may be equally effective, but the cost in water consumption and in rehandling, or in combining a washing system in the plant process, is probably greater and wet aggregate will not screen so effectively. The fine silica dust removed in this plant is so hard it has no small value as a grinding powder. In crushing from quarried rubble there can be no earthy, clayey, or vegetable matter in the aggregate, and this, together with the removal of all fine stone dust, may be said to be the



Stone-crushing Plant before Roofing Over.
The feed-shutes from C to D and D to E were removed at the time the photograph was taken.



THE CONCRETING YARD AT BOMBAY, LAID OUT ON MASS PRODUCTION LINES.

first point in the high quality of concrete obtained.

CONCRETING.

In the concreting shop pressed steel moulds with spring-closing clips are used for the sleeper blocks, which, in clipping, secure the guide and reinforcement in the proper position. These moulds are placed on framed trolleys which proceed in a continuous cycle through the various operations. The operations are:—(1) Placing moulds on trolleys; (2) cleaning moulds; (3) placing reinforcement and closing moulds; (4) filling in concrete and its vibratory consolidation; (5) removal of the moulds from the trolley to the setting ground; and (6) stripping off moulds and placing blocks in water to mature. After (5) the trolleys load up again with empty moulds and proceed around the cycle again.

For the aggregate four sizes of stone (⅛ in. and below, ¼ in., ½ in. and ¾ in.) are used and carefully graded so as to give as dense a concrete as possible. On the principle of Dr. Oscar Faber's recent experiments and data it is very nearly a 1 : 1½ : 3½ mix with 18 per cent. cement, which with vibratory consolidation gives a better concrete than usually obtains with 1 : 1½ : 3 concrete in the best reinforced structural work. So dense, sound, and hard is the finished product that a sleeper block almost rings like metal in one of the final tests for soundness.

The full arrangement of concreting plant, with controlled chute-feed to each mould, giving the minimum of handling of material, has not yet been completed due to difficulties in getting the plant required. When complete the tip wagons bringing stone aggregate from the crushers will pick up cement and discharge the batch into the skip elevators feeding the mixers. The mixed batch will empty into chutes leading to hoppers fixed over the trolleys as they stand on the vibrating tables, the hoppers having tubes leading down to just above where the mouths of the moulds standing on the trolley come to rest. The tubes have control gates, and when two trolleys of moulds have come up on to the vibrating tables vibration is started and the concrete is let in gradually and well shaken down and consolidated as the moulds fill. The mould closing-plates are then dropped in position and the trolleys are shunted down to the setting-ground line, other trolleys coming up from the assembling shed to the vibrating tables. On full output two trolleys carrying sixteen moulds come on and leave each vibrating table every four to five minutes, thus producing, say, two sleepers per minute, and this is the speed of production maintained in every shop throughout the process of making each component part.

The illustration of the concreting depot shows, from left to right, the assembling shed, the setting ground with its feed line, the maturing tanks, and the despatch platform with sleeper blocks stacked thereon and two wagons loading up. The excavations in the middle foreground are in connection with the concrete plant extensions. The photograph was taken early in the day's production, when the setting ground was only covered at the nearer end with the moulds to set. By the end of each day the setting ground is covered to the far end, the blocks at the nearer end having been stripped and the moulds taken into use again and those set having been put into the maturing tanks.

The moulds are left on the setting ground for from 4 to 6 hours before they are stripped off the blocks, and the blocks are put into the maturing tanks some three hours later, or in any case from 12 to 24 hours after being cast. They lie under water in the tanks for 28 days before they are removed to the plugging ground, which is also the despatching platform.

It will thus be seen that the factors which go to produce an excellent quality concrete are: (1) The aggregate is scrupulously clean hard stone, carefully graded to get the greatest density with all sizes of aggregate; (2) the concrete receives vibratory consolidation; (3) it is placed in steel moulds that cannot cause internal stresses in the half-set blocks by warping or contracting; and (4) the concrete is matured under water for 28 days.

The quality of the concrete is up to a standard not often obtained, but in sleepers a high standard is essential if they are to stand up to modern 150-ton engines travelling at 60 miles an hour.

THE "STENT PLUG."

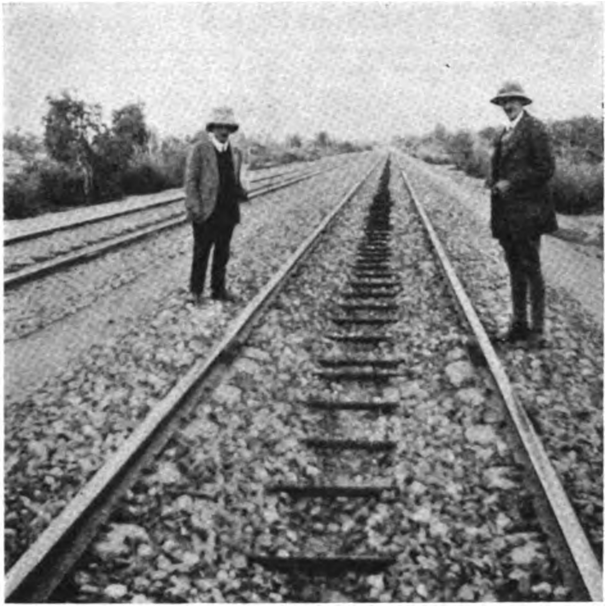
In the timber shop where the plugs are made the method of progressive kiln

"STENT" SLEEPERS ON THE B.B. & C.I. RAILWAY.

seasoning and the automatic machines for shaping the plugs from the dried "sticks" are interesting, but it is chiefly the holding power of the plugs and the method of setting them in the concrete that will interest concrete constructional engineers. The secret of the holding power lies in the formation of the hole into which the dried and treated plugs are driven, but the drying, seasoning and the treatment of the plugs are equally important. The wall of the hole is corrugated helically, and a steel wire coil is set in the concrete so that it forms the crest of each corrugation. Thus on driving the prepared and dried plug it bears only upon the steel, and, being tapered, is compressed in the driving. It soon takes up atmospheric moisture and expands between and into the corrugations, thus securely keying itself in. The forming of the holes and the setting of the coils is done in a simple manner by the use of metal false plugs, corrugated on the surface, on to which the machine-made steel reinforcing coils are screwed. The false plugs are then keyed into the mould or shuttering, and the concrete placed around them in the usual way. When the concrete has set the false plugs are easily screwed out, automatically leaving the coil in the correct position. Plugs can be driven in very rapidly and are secure within an hour of being driven. The plugs cannot split as they are bound in with the steel coils, and they cannot become loose because they cannot become drier (or therefore shrink smaller) than when driven, no matter what the climatic conditions may be.

TESTS.

The Government of India Railway Board with various other railway engineers recently visited Messrs. Bird & Co.'s works at Delhi and witnessed a demonstration of the processes of manufacture and some tests on the sleepers and the plugs. A test of the holding power of the plug in a sleeper showed the withdrawing force required to start movement on an ordinary straight-sided dog spike to be 6,100 lb. or about double that required for a similar spike driven into an ordinary type of hard-wood sleeper. It was stated that with a harder wood plug results of from 9,000 to 11,000 lb. had been obtained. A spike was also shown which had been driven into a plug and the head twisted nearly off, showing how the plug held



A Six-Mile Length of "Stent" Sleepers near Samasatta, N. W. Railway.

Some goods wagons were derailed and dragged over this track, but none of the concrete blocks was broken or had to be replaced.

the spike almost as a vice. The cutting out of plugs for renewal was also demonstrated.

An impact test was applied to a "Stent" concrete sleeper for 19-ton axle loads and to a heavy D.O., E.I. Rly., cast-iron plate sleeper for 20-ton axle loads. The iron sleeper broke up entirely under a test load that only produced two cracks in the concrete sleeper, the rail fastenings of which remained tight and intact. A test run over a piece of road laid with these sleepers was also made, and the smooth running and quietness of the road were clearly demonstrated.

The N.W.Ry. are believed to have 60 miles of these sleepers laid and will shortly have over 100 miles. The majority of the big Indian railways have some of these sleepers under trial, and with the growing output the manufacturers hope to be able to supply a larger number of railways.

Mr. Stent, of 6E, Hyde Park Mansions, London, N.W.1, has patented this sleeper in practically every country, and would be pleased to give further information on the subject to those interested.

[The "Green-Moore" concrete sleeper, which is also being adopted in India, was illustrated and described in our issue for December, 1922.]

TORSIONAL STRENGTH OF RECTANGULAR CONCRETE SECTIONS.

A RECENT issue of *The American Architect* publishes an article containing some interesting data on the torsional strength of plain and reinforced rectangular sections of concrete, by Messrs. C. R. Young (Associate Professor of Structural Engineering, University of Toronto), and W. L. Sagar and C. A. Hughes (Demonstrators in Applied Mechanics at the University of Toronto). The authors state :—

Cases of torsion on rectangular sections of concrete or reinforced concrete are of frequent occurrence in practice, and under certain conditions the stresses resulting therefrom may be large. Wall beams, girders or headers next to floor openings, wall foundation beams used in cantilever wall footing construction, and piers supporting skew arches are examples of members subjected to twisting or torsional moment. More commonly than not, any analysis of the stresses in members subjected to twisting has been made by the simple torsion theory, which applies only to circular sections, with the result that large errors have often been committed in the calculations. While for square sections the true maximum shearing stress is only 6 per cent. greater than is given by applying the common or circular section theory, for a rectangle with the long side twice as great as the short one, the true maximum stress is 50 per cent. greater than is given by the common theory.

EXACT METHOD.

Exact analysis of the torsion problem was first made by the French elastician, Saint-Venant. The expression for torsional shearing stress derived by him is very cumbrous. Perhaps the most remarkable conclusion reached by Saint-Venant was that the maximum torsional shearing stress does not arise at the corners of a rectangular section, as was implied in the common or older theory, but at those points on the boundary nearest the centroid, namely, at the centres of the long sides.

For practical purposes of design it is useful to employ a simplification of Saint-Venant's formula proposed by R. J. Woods in " Strength and Elasticity of Structural Members." His formula is:

$$T = kb^2hq_1$$

where k has values varying with the ratio of h to b as follows :

h/b	k	h/b	k
1	0·208	3·5	0·275
1·5	0·231	4	0·282
2	0·246	5	0·292
2·5	0·258	10	0·312
3	0·267		

In this formula, T is the torsional moment in inch-pounds ; b is the short side and h the long side in inches, and q the maximum torsional shearing stress at the centres of the long sides.

TORSION ON REINFORCED CONCRETE BEAMS.

Since, from the nature of torsion, elongation of the edges of a rectangular section sets up, it might appear that longitudinal reinforcement in the corners of a concrete beam subject to torsion would be useful. In the lower corners of most beams longitudinal rods running full length already exist, and these might conceivably assist in resisting the warping effect of torsion, as might top corner rods carried full length of the beam.

TORSIONAL SHEAR REINFORCEMENT.

Provision for torsional shearing stresses on a given cross section should apparently be made in very much the same way as it is made for ordinary shearing or diagonal tension stresses. The inferior strength of concrete in tension as compared with shear naturally tends to bring about rupture of a torsion specimen of concrete along diagonal lines, and this is substantiated by actual experiment. Investigation has shown that rupture of concrete specimens in torsion is along a spiral surface inclined at 45 deg. with the axis. It is, therefore, reasonable to assume that spiral reinforcement at an angle of 45 deg. would be most effective, just as web reinforcement for vertical shear in beams is found to be best if inclined at this angle. Inclined stirrups or bent up rods in beams placed there for the purpose of resisting vertical shear, or diagonal tension, have some value, so far as they go, in resisting torsion. For torsional shear, however, they should run in opposite directions on opposite faces of the beam. On the side next the loading the usual bent up steel is available for the resistance of both vertical shear and torsion, but on the far side it runs in the wrong direction to resist torsional stresses. However, as the existing vertical shearing stresses operate against the torsional stresses, the steel on the far side may not be required at all for torsional stresses. Some designers have, for beams carrying a heavy torsional moment, placed all of the inclined steel on the inside of the beam, so that the combination of the vertical shearing and torsional shearing stresses on this side might be met effectively. Such practice assumes that on the far side the two kinds of shearing stress offset each other to such an extent that the net shearing stress is within the capacity of plain concrete.

For very heavy torsional stresses, additional reinforcement may be provided by placing steel in the top and bottom faces inclined to the axis of the beam, at, say, 45 deg. In such cases the natural system of reinforcement to employ would be heavy wire, or light rods wrapped about the longitudinal steel at a 4 deg. spiral with a pitch such that at least one inclined rod would be cut on each face of the beam.

Although great care is customarily taken by designers of reinforced concrete to ensure that shearing stresses in beams do not rise above a conservative specified limit, as, for example, 120 lb. per sq. in. under the Joint Committee specifications, torsional shearing stresses may, in certain instances, amount to over 100 per cent. of the vertical shearing stresses. Wall girders of comparatively short span supporting flexible beams of long span from the interior of a building may be loaded with concentrations of 10,000 lb. or more on one side only. The torsional moment thus produced on a wall girder of, say, 10 by 20 in. section would, assuming the torsion to be wholly absorbed by the girder, give rise to a maximum torsional shearing stress on the long sides of the girder of approximately 100 lb. per sq. in., or nearly as much as the limiting vertical shearing stress.

That no failures have been recorded through the neglect of torsion in designing rectangular beams, is no doubt due to the restraining effect of the slab and to the fact that the working stresses for shear heretofore employed have been very conservative. With the growing tendency to increase shear stresses to as much as 12 per cent. of the ultimate compressive strength of the concrete, greater care will need be taken to provide for torsion as well as for shear.

TESTS ON CONCRETE SECTIONS.

But little investigation of the torsional strength of concrete appears to have been carried out either on this continent or abroad. Mörsch cites torsional tests to failure on a series of solid and hollow cylindrical specimens of plain concrete and on some hollow cylinders reinforced by spirals.* The solid cylinders, which consisted of 1 : 4 concrete from 79 to 98 days old, gave an average computed torsional shearing stress of 243 lb. per sq. in. Hollow cylinders of the same material, from 52 to 55 days old, gave an average strength of 247 lb. per sq. in. Addition of 45 deg. spiralling to cylinders, tested when 60 days old, gave computed ultimate torsional shearing strengths of from 513 to 736 lb. per sq. in., depending upon the amount of reinforcement. The increment in strength over that of the plain cylinders was in direct proportion to the reinforcement and ran from 31 per cent. to 124 per cent.

Experiments carried out for the Deutscher Ausschusz für Eisenbeton comprising solid cylindrical, hollow cylindrical, square and rectangular specimens of 1 : 2 : 3 mix, 45 days old, gave ultimate torsional shearing stress of from 243 to 462 lb. per sq. in. with an average of 375 lb. per sq. in. The reinforced specimens included those with longitudinal rods only and some with longitudinal rods and spiralling. It was found that longitudinal reinforcement was in itself of little value, improving the resistance of the specimen by not more than 12 per cent., even though rods were used in both the middle of the sides and in the corners. Sloping longitudinal rods were found to be of some value, providing they slope in the proper direction to resist the torsion. Spiralling to the extent of 0·5 per cent. provided with liberal longitudinal reinforcement increased the strength of specimens from 129 to 156 per cent.

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* See *Concrete and Constructional Engineering*, September, 1923.

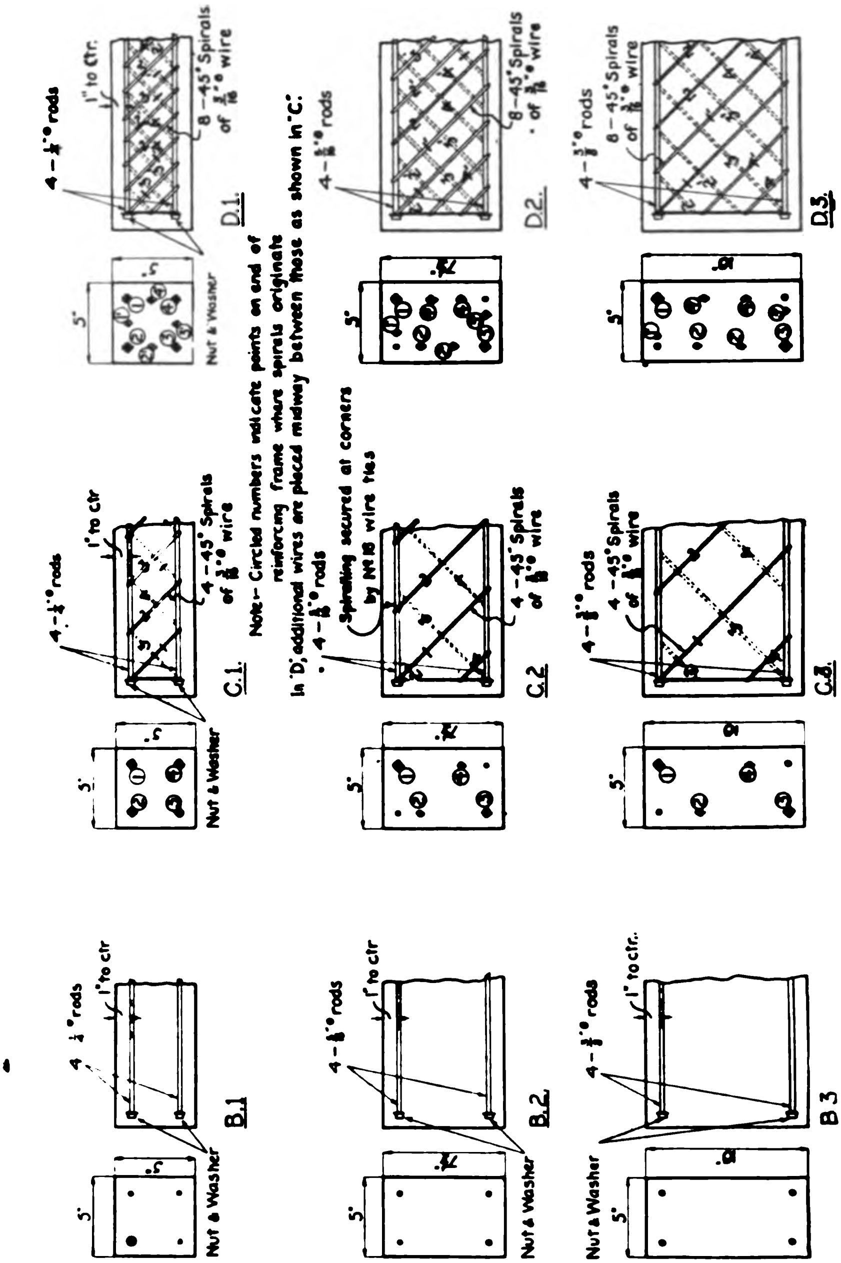

5"
5"
4 - ½"ø rods
1" to ctr
Nut & Washer
B.1
4 - ⅝"ø rods
1" to ctr
5"
7½"
Nut & Washer
B.2
4 - ⅝"ø rods
1" to ctr.
Nut & Washer
5"
10"
B 3
4 - ½"ø rods
1" to ctr
4 - 45° Spirals of 3/16"ø wire
Nut & Washer
C.1
Note:- Circled numbers indicate points on end of reinforcing frame where spirals originate
In 'D', additional wires are placed midway between these as shown in 'C'.
4 - ⅝"ø rods
Spiralling secured at corners by No 18 wire ties
4 - 45° Spirals of 3/16"ø wire
C.2
4 - ⅝"ø rods
4 - 45° Spirals of 3/16"ø wire
C.3
4 - ½"ø rods
1" to ctr.
8 - 45° Spirals of 3/16"ø wire
D.1
4 - ⅝"ø rods
8 - 45° Spirals of 3/16"ø wire
D.2
4 - ⅝"ø rods
8 - 45° Spirals of 3/16"ø wire
D.3

FIG. 1.—RECTANGULAR TORSIONAL SPECIMENS OF REINFORCED CONCRETE.

TABLE 1—SIZE AND REINFORCEMENT OF RECTANGULAR TORSIONAL SPECIMENS OF CONCRETE

Specimen No.	Nominal Size In	Actual Vert X Hor'l In.	Longitudinal Reinforcement	Spiral Reinforcement	Percentage of Spiralling
A1	5X5	4.98X5	None .	None	None
A2	5X7.5	5.0X7.5	"	"	"
A3	5X10	5.0X10.09	"	"	"
B1	5X5	5.06X5.0	4 rods ¼ in. dia.	"	"
B2	5X7.5	5.06X7.59	4 " 5/16 in. dia.	"	"
B3	5X10	5.0X10.19	4 " ⅜ in. dia.	"	"
C1	5X5	5.13X5.19	4 " ¼ in. dia.	4-45° spirals of 3/16" iron wire	0.94
C2	5X7.5	5.13X7.68	4 " 5/16 in. dia.	"	0.55
C3	5X10	5.18X10.31	4 " ⅜ in. dia.	"	0.39
D1	5X5	5.19X5.31	4 " ¼ in. dia.	8-45° spirals of 3/16" iron wire	1.88
D2	5X7.5	5.19X7.81	4 " 5/16 in. dia.	"	1.10
D3	5X10	5.13X10.38	4 " ⅜ in. dia.	"	0.78

UNIVERSITY OF TORONTO TEST SPECIMENS

In planning the investigation here reported, it was thought desirable to make up the test specimens so as to throw light on the following :

(1) Effect of varying ratio of the long and short sides of the cross sections.

(2) Effect of adding longitudinal reinforcement only.

(3) Effect of adding longitudinal reinforcement and light spiralling.

(4) Effect of adding longitudinal reinforcement and heavy spiralling.

(5) Value of torsional modulus of elasticity of plain concrete.

Twelve test beams in all were made. They were of three sizes, 5 by 5, 5 by 7½ and 5 by 10 in., all beams being 5 ft. long. For each size, one beam was made of plain concrete ; one contained light longitudinal reinforcement only ; one contained medium longitudinal reinforcement and light spiralling ; and a fourth heavy longitudinal steel and close spiralling. Details of the reinforcement are given in *Table* 1 and in *Fig.* 1.

The specimens were made of 1 : 2 : 4 concrete and were tested at the age of 28 days. The mild steel longitudinal reinforcement had a yield point varying from 34,600 to 37,200 lb. per sq. in., while the yield point of the spiralling wire was 31,550 lb. per sq. in.

Plain concrete specimens when broken in torsion gave the characteristic spiral fracture shown in *Fig.* 2. The first evidences of failure in reinforced specimens were diagonal cracks crossing the faces of the beam at approximately 45 deg.

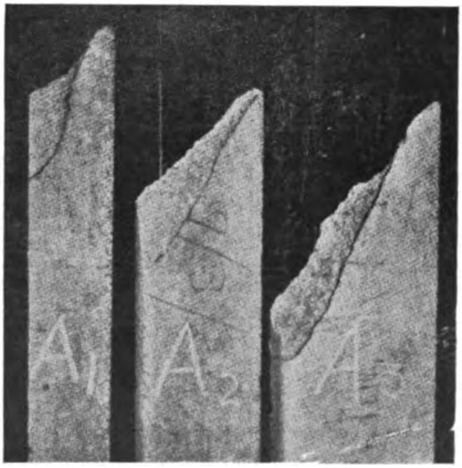
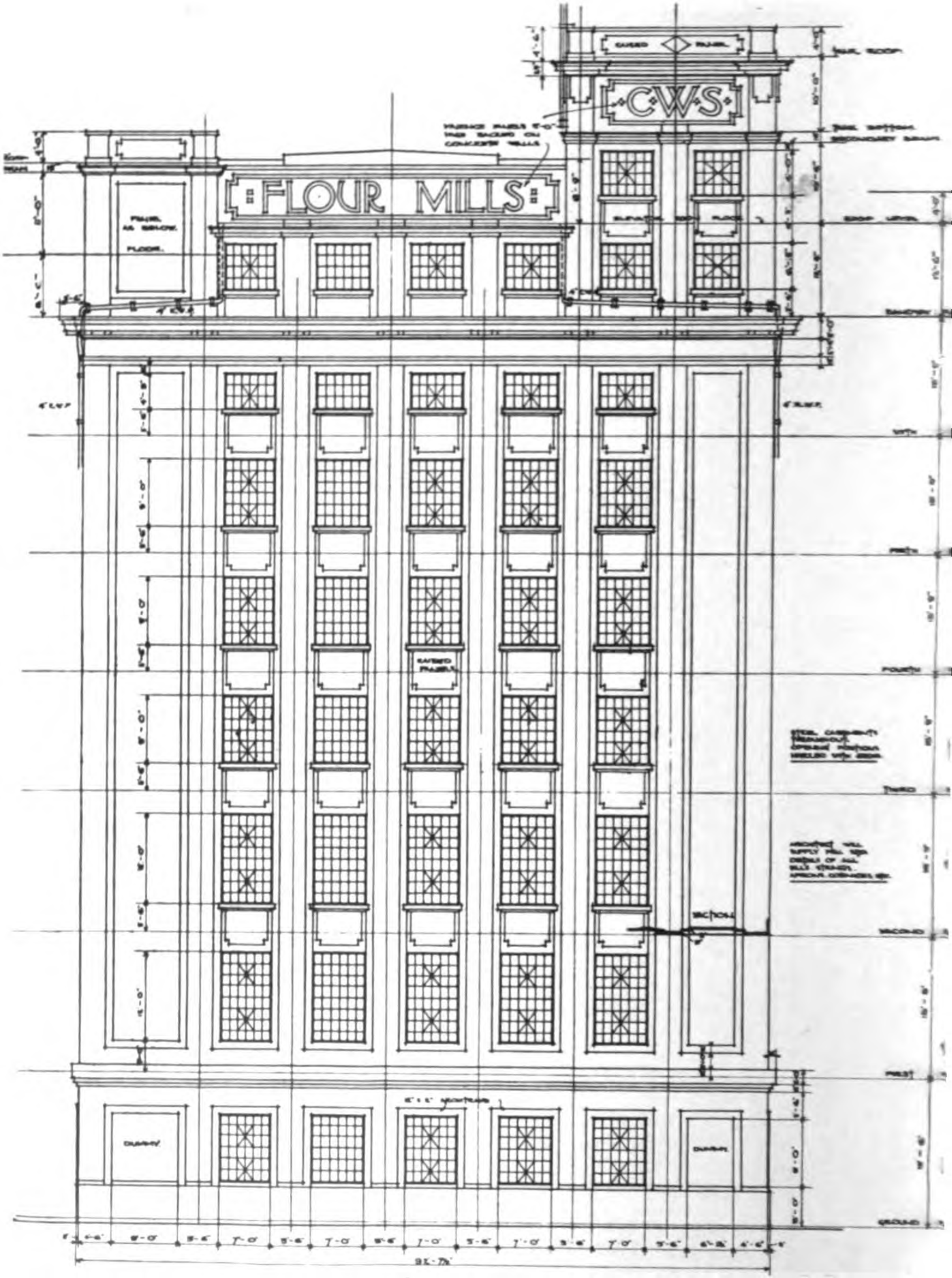


FIG. 2.—TORSIONAL FRACTURE OF RECTANGULAR SPECIMENS OF PLAIN CONCRETE.

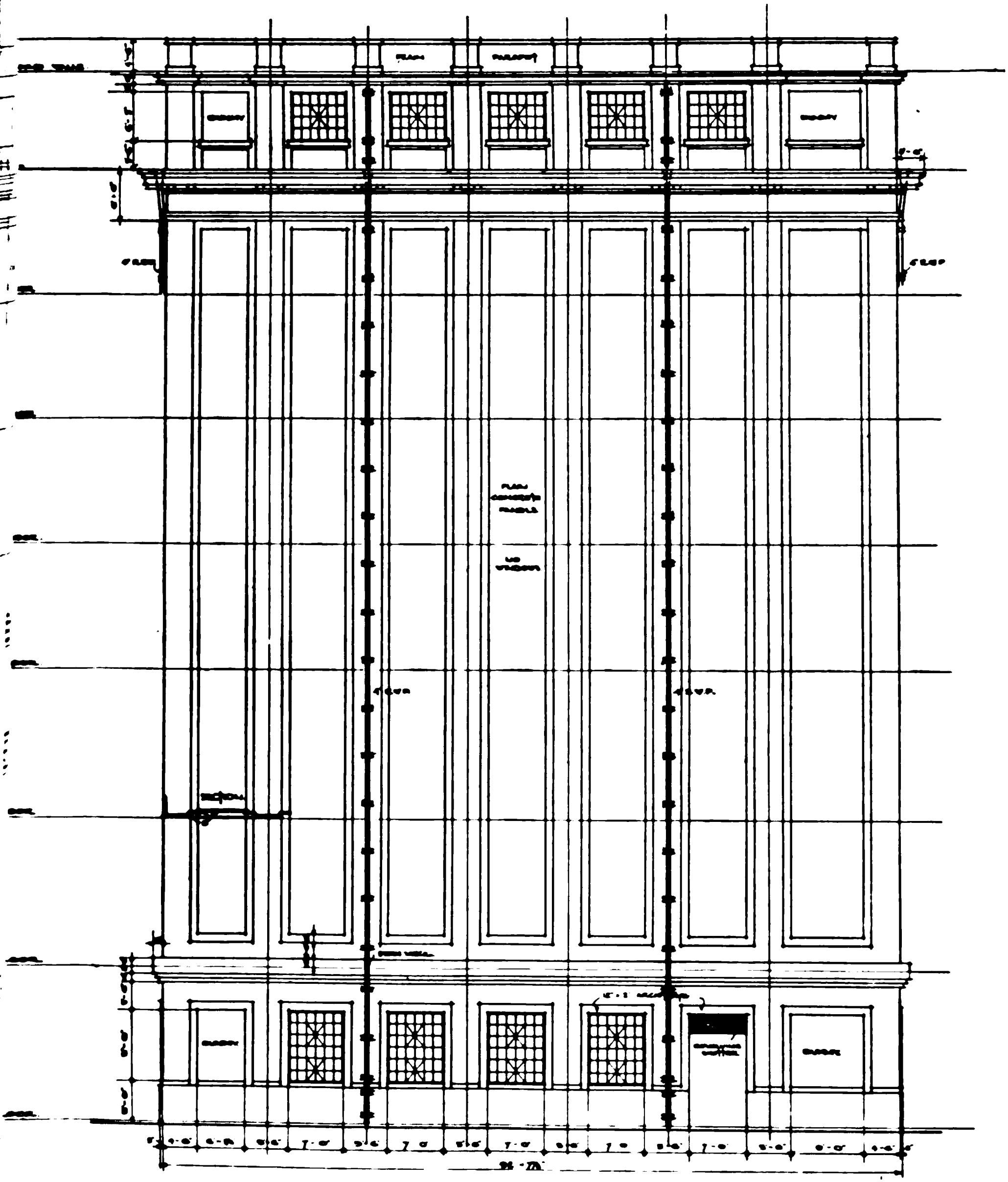
(*Continued on page* 680.)



East Elevation.

REINFORCED CONCRETE GRAIN SILOS AT AVONMOUTH, FO

[The North and South Ele



WEST ELEVATION. [*Mr. L. G. Ekins, F.R.I.B.A., Architect.*

ɛ THE CO-OPERATIVE WHOLESALE SOCIETY, LTD. (*See p.* 641.)
ations are given on pp. 646–7.]

(*Continued from page* 677.)

with its axis. These cracks, for the most part, originated near the centre of the span, where by reason of the combination of the torsional and flexural stresses they would be most likely to start.

TEST RESULTS.

The three plain concrete test beams A1, A2, and A3 were made up and tested (1) to discover the torsional strength of plain concrete, and (2) to determine the torsional modulus of elasticity of this material. The torsional moment under which each of these members failed, together with the apparent or computed maximum torsional shearing stress for both plain and reinforced specimens is given in *Table* 2. This apparent fibre stress at failure was calculated by applying the exact torsion theory to these rectangular cross sections on the assumption that the material was homogeneous and isotropic and that the proportionality of stress and deformation obtained up to the moment of failure. For rectangular cross sections, the maximum torsional shearing stress which arises at the centre points of the long sides of a rectangle, or at the centre points of each of the four sides of a square, may be conveniently expressed by the previously cited formula of R. J. Woods, rearranged:

$$q_1 = T/kb^2h$$

A comparison of the relative strength of torsional members variously reinforced may be made by considering the figures in the fifth column of *Table* 2. At the same time the value of each type of reinforcement may be seen by comparing the strength of the member in which it occurs with the strength of a plain concrete member of the same size.

From the fourth column of *Table* 2 it is seen that the average computed maximum torsional shearing stress for the plain concrete specimens is 536 lb. per sq. in. Since the tensile strength of

TABLE 2—ULTIMATE TORSIONAL SHEARING STRENGTH OF RECTANGULAR BEAMS

Specimen No.	Actual Ratio $\frac{h}{b}$	Torque at Failure in.-lb	Computed Maximum Torsional Shearing Stress, lb. per sq. in	Ratio to Average Torsional Strength of Plain Concrete Specimens	Average, or Secant Torsional Modulus of Elasticity, F, lb. per sq in.
A1	1.00	13.900	539		1.580.000
A2	1.50	20,200	467		1,625.000
A3	2.02	37,400	602		1,352 000
			Average 536		Average 1.519.000
B1	1 01	14.000	532	0.99	
B2	1.50	22,700	506	0.95	
B3	2.04	36,700	584	1.09	
			Average 541	Average 1 01	
C1	1.01	19,000	668	1.25	
C2	1.50	29,000	621	1.16	
C3	1.99	36.000	530	0.99	
			Average 645 omitting C3	Average 1.21 omitting C3	
D1	1.02	25.000	838	1.56	
D2	1 50	36.000	740	1.38	
D3	2.02	53,400	790	1.47	
			Average 789	Average 1 47	

TABLE 3—STRESS IN SPIRALLING AT FAILURE OF SPECIMENS

Specimen No.	Excess of Torsional Moment Over that for Corresponding Plain Concrete Specimen, in.-lb.	Stress in Spiralling lb. per sq. in.
C1	5,100	18,000
C2	8,800	31,000
C3	Unreliable	——
D1	11,100	19,500
D2	15,800	27,900
D3	16,000	28,200

1 : 2 : 4 concrete varies from about 180 to 440 lb. per sq. in., the torsional approaches the tensile strength more nearly than the shearing strength. This is to be expected when it is remembered that at any face of the specimen the stress condition resembles that in the web of a concrete beam ; that is, the governing stress is diagonal tension.

A consideration of the results for the specimens of series B, which were reinforced with longitudinal steel only, shows that corner reinforcement has apparently little value, since the computed stresses developed at the points of maximum stress average but little higher than for unreinforced specimens. The same general principle was disclosed in the Deutscher Ausschusz für Eisenbeton experiments, although side rods added materially to the strength of the specimens in the latter tests. The angular distortion of a concrete beam is not great enough up to the point of failure to bring corner longitudinal reinforcement into play as a factor in resisting the applied torque.

Some benefit is derived from spiral reinforcement, as will be seen by considering the results for the specimens of series C and D. The wide pitched spiralling of series C does not, however, strengthen the member to the extent that might be expressed, the strength as compared with a plain concrete specimen being only, on an average, 21 per cent. more, excluding in the average the value for beam C3 which obviously is erroneous and unreliable.

The spiralling of series D, being only half the pitch of that of series C, might be expected to strengthen the members proportionately. The addition averages 47 per cent. of the strength of the plain concrete members, or somewhat more than double the percentage increase for series C.

Considerably less benefit from spiral reinforcement is indicated in the series of tests here reported than was found in the tests for the Deutscher Ausschusz für Eisenbeton. With 8 spirals, giving a spiral reinforcement of about 0·5 per cent., the latter tests gave an increase in strength of from 129 to 156 per cent. In the latter tests there were from 6 to 8 longitudinal rods employed and the size of specimens was considerably greater than in the tests carried out by the authors. Doubtless the use of longitudinal rods in the middle of the sides of the German specimens contributed substantially to their torsional strength. The greater size of the specimens at the same time rendered the irregularities of the fabricated reinforcement units much less important. Particularly with the 5 by 5 in. University of Toronto specimens the difficulty of securing a reinforcement unit to full plan dimensions at the centre cross section was great. However, no better work could be expected in actual practice, and probably it would be much worse. It is not known whether special care was taken in making the German specimens, but the authors' specimens were made under typical commercial conditions. Attachment of the spiralling to the side rods may have further accounted for the high strengths given in the German tests.

Computation of the stresses in the spiralling of the specimens of series C and D shows, according to *Table* 3, that the breakdown is not necessarily coincident with the passing of the yield point, which was 31,550 lb. per sq. in.

For the 5 by 7½ in. and 5 by 10 in. specimens, however, the yield point in the spirals was closely approached at the point of failure.

Mörsch's method of calculating the stress in the spiralling was adopted. According to this the steel spirals are assumed to develop the difference between the ultimate torsional moment of the reinforced specimen and the ultimate torsional moment of a corresponding plain concrete specimen. Half of the shearing resistance is assumed to be taken up by the reinforcement which lies at an angle of 45 deg. to the torsional shear plane, and half by the compressive resistance of the concrete acting at right angles to the direction of the reinforcement.

The value of closer spiralling in series D is evident from an examination of *Fig.* 3. Where four spirals only are used, as in (*a*), there is a longitudinal zone of length *DS*, in which a transverse plane will cut only one diagonal wire or rod on the near face of the specimen. Further, there is a depth *D'S'* in which torsional shearing stresses may act on plain concrete. The corresponding distances for a specimen with eight spirals are shown in (*b*), and are very much less. As much smaller zones are left unreinforced with close spiralling, its advantage is evident.

Inclined stirrups or bent up rods in a reinforced concrete beam may be counted upon, if their area is in excess of ordinary shear requirements, to help resist the torsional shear on the face next the loading. On the other face, unfortunately, they run in the wrong direction. If the net shearing stress on this face due to the combination of vertical shear and torsion be in excess of the shearing strength of plain concrete, special torsional reinforcement would need to be added. Anchorage for such steel is most readily provided by utilizing it in the form of a spiral, thus increasing the torsional resistance through the presence of steel on the horizontal faces.

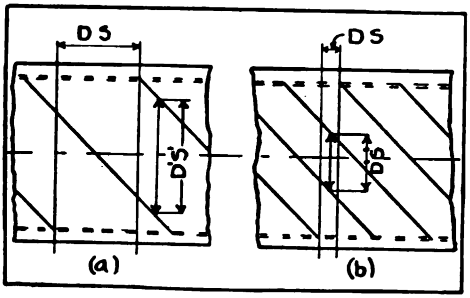


FIG. 3.—EFFECT OF PITCH FOR SPIRALLING ON TORSIONAL STRENGTH.

TORSIONAL RIGIDITY.

The angular deformations observed for the various specimens are plotted in *Figs.* 4, 5 and 6. In these diagrams the degrees of twist in a length of 40 in. are shown for various torsional moments nearly up to the point of failure.

Examination of the curves shows that the rigidity of a specimen of any given size is but little increased by the addition of reinforcement, whether it be longitudinal or spiral. Such deformation as can occur before the breakdown of the specimen is apparently not affected by reinforcement of the type and quantity used in the specimens.

For plain concrete specimens the torque-twist curve is substantially straight. The average or secant torsional modulus of elasticity, *F*, has been computed for the portion of the curves up to practically the last plotted points and is given in the last column of *Table* 2. The formulæ used were as follows :

$F = 410\ Tl/ab^4$, for square specimens.
$F = 205\ T(b^2 + h^2)/ab^3h^3$, for rectangular specimens.

Where T = torque in in.-lb.
l = length of specimen in inches between gauge points
a = angle of twist in degrees.
b = short side in inches.
h = long side in inches.

The curvature of the torque-twist curves for the spirally reinforced specimens is more marked than for the plain specimens for those containing only longitudinal reinforcement. For those containing the close spiralling the addition to the toughness of the specimen is marked.

CONCLUSIONS.

Study of the behaviour of rectangular beams of concrete under torsion, as evidenced by the specimens tested by the authors, appears to warrant the following conclusions :

(1) Rectangular specimens of plain concrete with various ratios of long to short sides develop substantially the

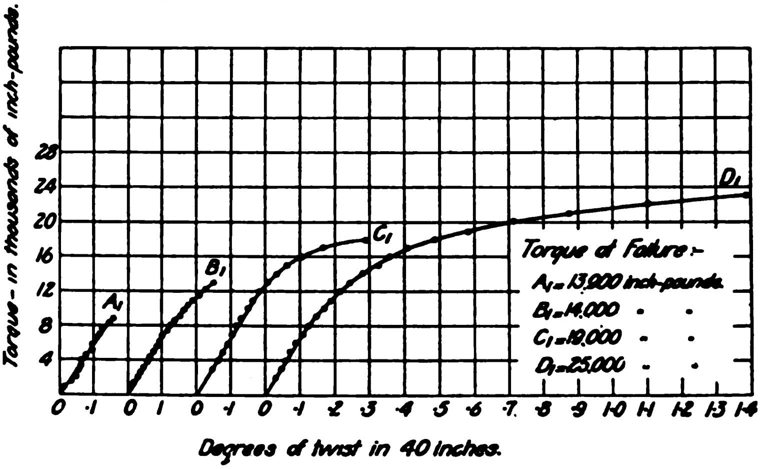


Fig. 4.—Torque-Twist Curves for 5 in. by 5 in. Specimens. Plain and Variously Reinforced.

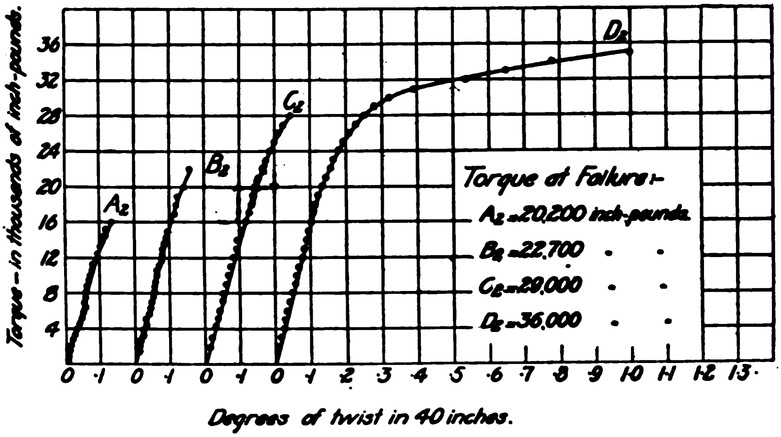


Fig. 5.—Torque-Twist Curves for 5 in. by 7½ in. Specimens.

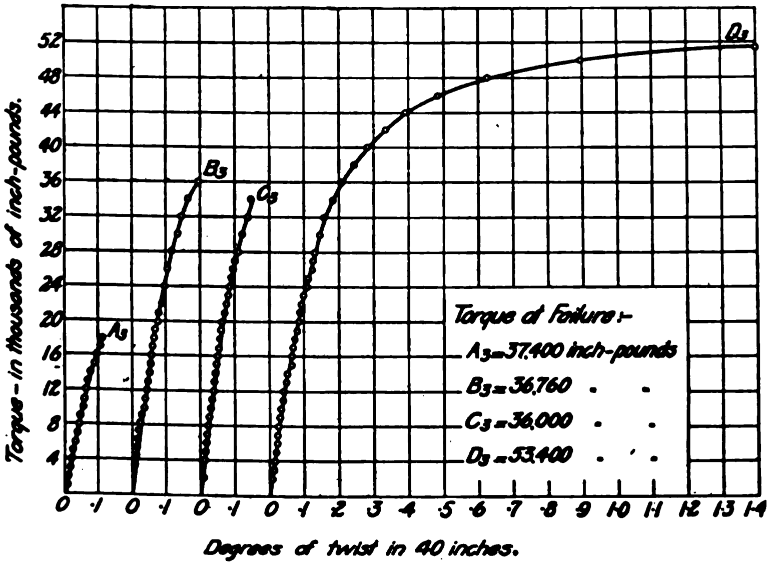


Fig. 6.—Torque-Twist Curves for 5 in. by 10 in. Specimens.

same computed maximum torsional shearing stress at the centre of the long sides. For 1 : 2 : 4 concrete one month old this is about 540 lb. per sq. in.

(2) Longitudinal reinforcement alone is of little value in reinforcing concrete beams for torsion.

(3) Spiral reinforcement increases the torsional strength of beams, the increment in strength being approximately proportional to the amount of such reinforcement. Spiralling to the amount of from 0·4 to 1·0 per cent. may be expected to increase the torsional strength of a member with or without straight longitudinal rods from 20 to 50 per cent.

(4) The torsional modulus of elasticity of plain concrete of 1 : 2 : 4 mix one month old averages about 1,500,000 lb. per sq. in.

BOOK REVIEW.

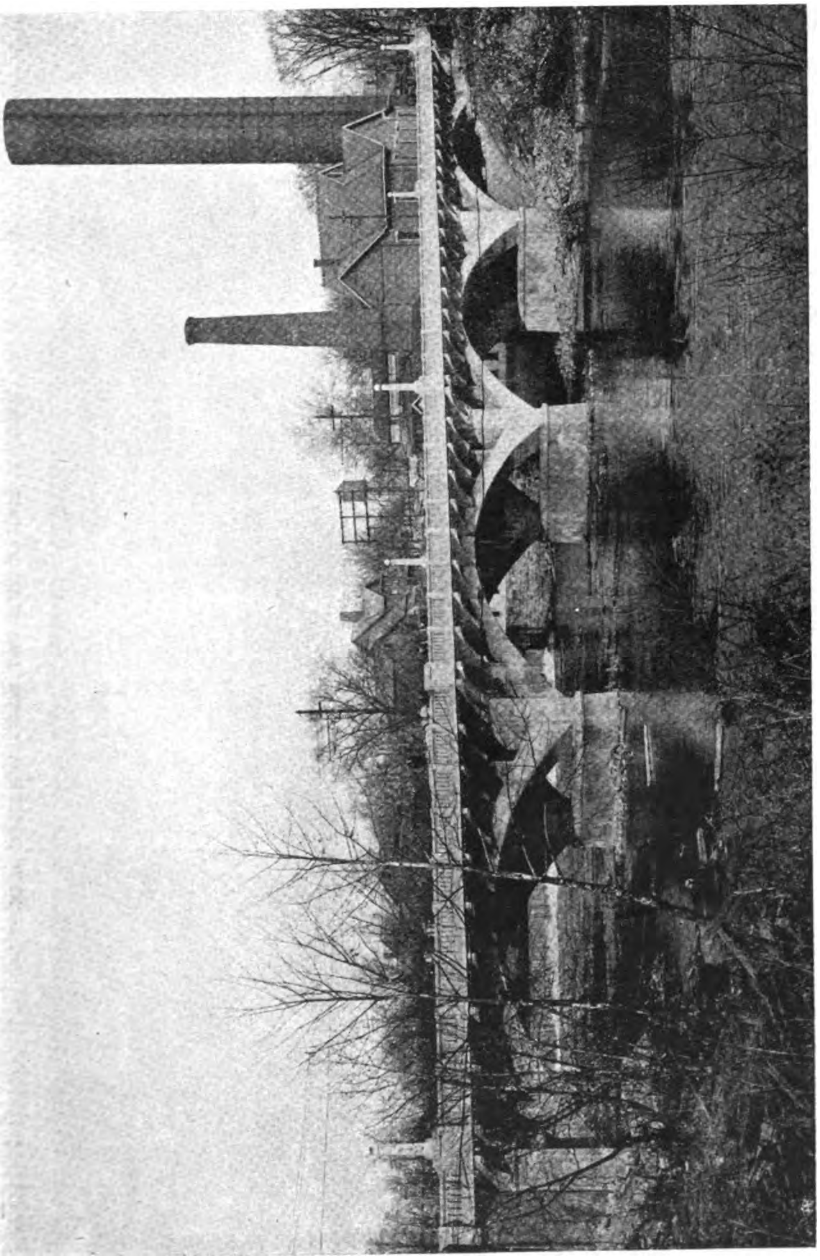
The Load-bearing Power of Piles.

By Dr. H. Dorr (68 pp., with 61 illustrations).
Berlin: Wilhelm Ernst and Sohn. Price 270 marks.

MANY attempts have been made to ascertain the maximum load-bearing power of piles under various conditions, but the results have not been satisfactory. The author has followed the recent work of Stern and Kafkas, who omitted to consider several important factors. The author also bases his new formulæ on Engesser's " wedge " theory of equilibrium, and by these means develops a number of formulæ for piles of various shapes. Special consideration is given to the resistance of the piles to compression and tension, to the maximum length of the piles, to the pressure of the structures they support, and the effect of the nature of the ground in which the piles are immersed and its coefficient of friction. Some particularly useful notes are given on the effect of various methods of ramming, on the sinking of the piles under load, and on the spacing of the piles in any given area. The risk of the piles bending is briefly considered.

The chief conclusion reached by the author is that all previous formulæ for ramming piles are of no value and are apt to be misleading, because they do not take sufficiently into account the specific properties of the ground to be piled and the enormous increase in carrying power obtained by a short increase in the length of a pile ; in one case, the increase in the length of a pile from 16 ft. 6 in. to 20 ft. (i.e., an addition of only 3 ft. 6 in.), raised the load-bearing power from 20 tons to 31·4 tons, or an increase of 57 per cent., the lower end of each pile being in loose silt. Another important conclusion is that conical piles have a greater carrying power than those with parallel sides, but it is merely a waste of material to make the piles conical throughout ; the cone should extend from the base to a height which depends on the nature of the ground ; above this point, the pile may be of smaller diameter, and have parallel sides. A straight pile with an enlarged foot of the shape of an inverted cone is only of use, however, when it rests on solid ground. If driven into quicksand the loss of frictional resistance in the pile as a whole may be greater than that of a slightly tapered pile of the same length and upper diameter. The author regards a metal sheathing to the pile as undesirable, as the rougher the surface of the piles the greater will be their resistance to slipping when once they have been rammed into place. He regards longitudinal reinforcement as useless when piles are merely subject to load, but admits their value near the head of the pile. He is greatly in favour of piles produced *in situ*, and claims that his investigations show their superiority over rammed piles. Apart from this, the author's investigations confirm those of Bernhard, and show that the manner in which the pile is driven is a matter of small importance.

No less than thirty-four examples of different types are fully worked out, and as there is an illustrative diagram on every page the book is easily followed by any reader who is sufficiently acquainted with the German language.



REINFORCED CONCRETE BRIDGE AT WARREN, OHIO.

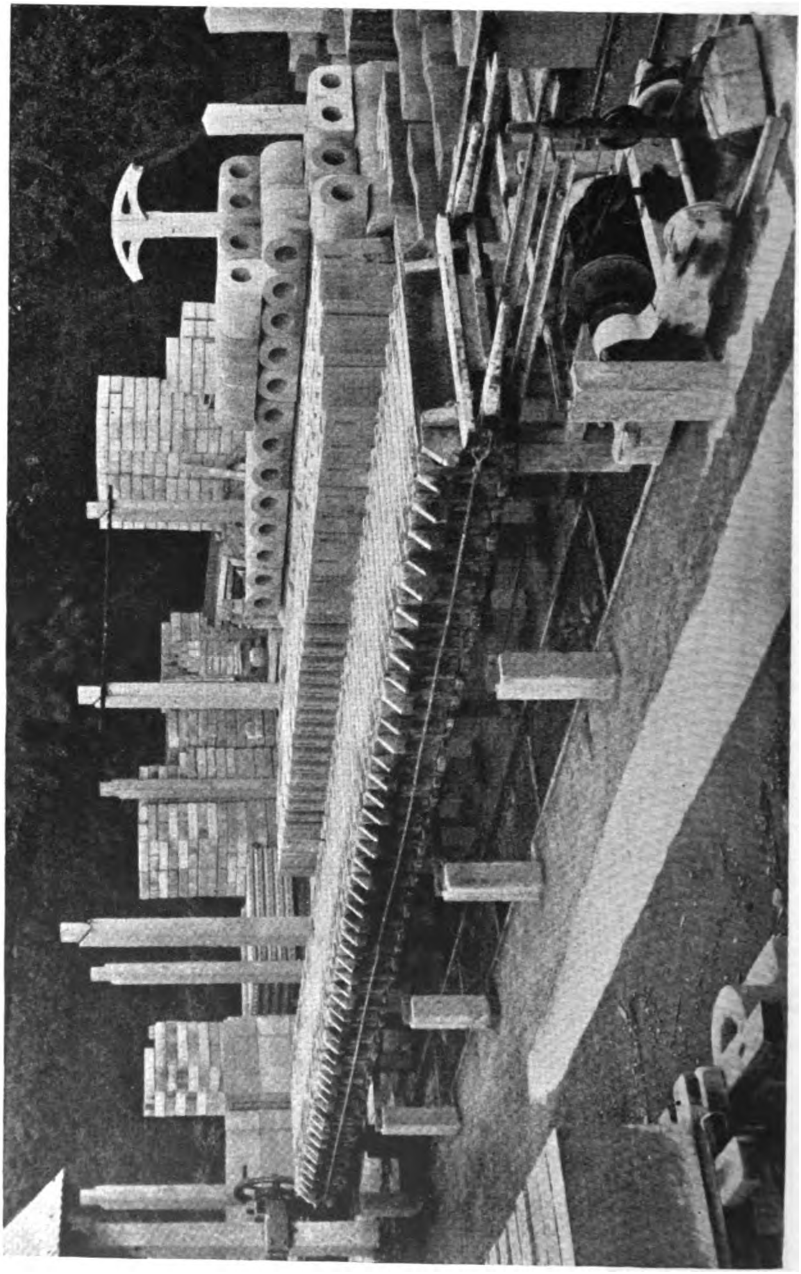


FIG. 4.—NEW CONCRETE BLOCK AND SLAB MAKING MACHINE. (See p. 687.)

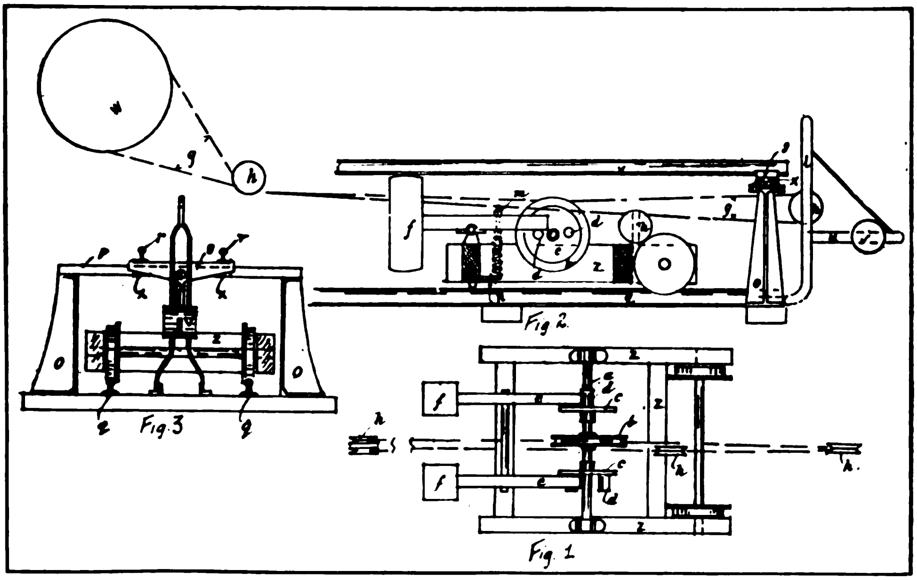
NEW CONCRETE BLOCK AND SLAB MAKING MACHINE.

We illustrate herewith a new concrete block making machine and shaker combined, which has the advantage of making blocks and slabs with a fairly wet mixture, and thereby obtaining the strongest concrete possible. This machine, which is the invention of Mr. W. Hopkins, of Leonard-Stanley, Glos., has been in use during the past eight months making fifty road kerbs per day for the Gloucestershire County Council.

The machine is of simple construction. It stands on legs 2 ft. high, carrying transverse steel channels, to which are attached rockers, mounted on trunnions on which rest longitudinal channels, forming a rack on which to place moulds to be filled with concrete for consolidation. It may be of a length to meet any required output.

The photograph (*Fig.* 4) shows the machine with a 30-ft. rack carrying moulds for fifty kerbs 2 ft. 6 in. by 12 in. by 5 in. The pallets and divisional boards rest on the channels forming the rack, and are clamped together by ½-in. rods running the whole length of each side of the machine. These are operated from one end by levers on pivots, so arranged that when the levers are pushed forward they tighten up the whole set of moulds and lock themselves in position; to discharge the moulds, all that is necessary is to release these rods by means of the levers, take the end boards away, and lift the pallet with kerb on to any suitable arrangement for removal. The next divisional board is then taken away and the kerb removed, and so on until all the moulds are empty. An extra set of pallets is required for the next batch of kerbs.

To facilitate ease and rapidity in handling the kerbs, which weigh 155 lbs. each, there is a rail track along one side of the machine, and pedestals for supporting loose timber frames on a level with the rack and of a length to hold 13 kerbs. On these the pallets with kerbs are placed on removal from the moulds. A trolley, with raising top, is run under this frame and the whole raised to clear the supporting pedestals. It is then run on to a side track, having similar supporting pedestals, on which this frame with the thirteen kerbs is lowered. The day's output, whatever it may be, is moved in this way, and remains on the side track until next day, when the blocks



Figs. 1, 2, and 3.—New Concrete Block and Slab Making Machine.

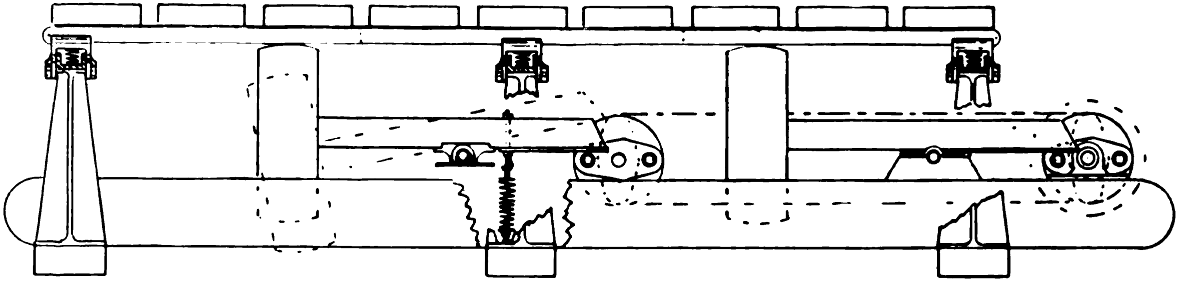


FIG. 5.

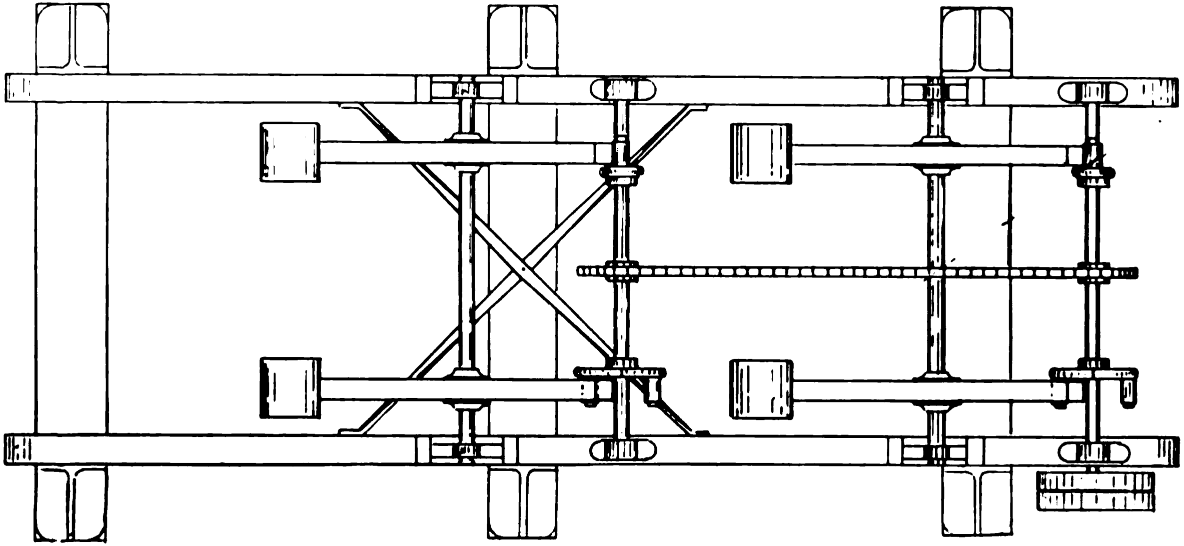


FIG. 6.

NEW CONCRETE BLOCK AND SLAB MAKING MACHINE.

are again moved in the same way for stacking.

The combined jarring and rocking motion—which is the special feature of the machine—is set up by tampers striking the underside of the rack, first on one side and then on the other. The impact of the tampers gives the jar or vibration, and at the same time slightly lifts the side struck, thus causing the rocking motion by the movement of rockers on the trunnions.

The power required is small, and one man can easily operate the machine.

After a few minutes' vibration the surplus water gradually rises to the surface of the concrete, without, however, carrying any cement or fine parts of aggregate with it, or in any way disturbing the mixture. The consolidation is so thorough that the kerbs in question are regularly removed the day after filling, and the moulds refilled. Those taken out one day are stacked the next, i.e., within forty-eight hours after making, thus effecting great economy both in space and in the time for watering and maturing, which is necessary with the semi-dry system. In the case of building blocks made on the site, it is stated that they can be used direct from the pallets in fine warm weather.

An interesting feature of the machine is the satisfactory way in which two or more layers of varying mixtures of concrete may be united, thus pointing to the possibility of making a compound block having a weather-resisting face and a porous back.

Another point worthy of note is that the usual cement film on the face of the block, which gives it the dull grey appearance so much objected to by many, can be removed with a hard brush and clean water at the time of taking off the pallets. This if done properly will bring out the natural characteristics of the aggregate used, and opens up great possibilities of using various mixtures for varying effects at a very small increase in cost.

It is claimed that in addition to other factors which enable good blocks to be produced cheaply by this machine, the absence of the necessity of ramming effects a saving of fully 25 per cent. in labour.

Figs. 1, 2 & 3 show the machine arranged with tampers mounted on traveller for moving to and fro and having power transmitted to them by wire rope drive. Any power allowing free movement of the traveller is suitable.

Fig. 1 is a plan of the traveller with shaft (*a*) carrying rope-driven pulley (*b*) and discs (*c*) with rollers (*d*) for lifting the tamper arms (*e*) and depressing tampers (*f*). A wire rope (*g*) is passed round the pulley (*b*) to give grip and then over and under the guide pulleys (*h*), as shown in *Fig* 2.

Fig. 2 is a side view of *Fig.* 1, with power pulley (*w*), wire rope (*g*) and tension bracket (*l*)—with pulley (*h*)—and balance weight (*v*) on arm (*u*). A coil spring (*i*) is fitted with an adjusting screw (*m*) passing through the arm (*e*) and held at the bottom by a hook (*n*). When the arm (*e*) is lifted by the roller (*c*) the spring is

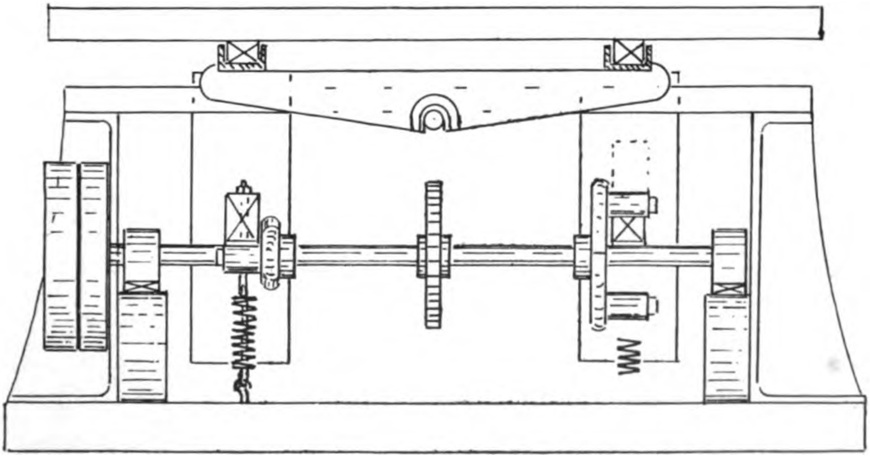


FIG. 7.—NEW CONCRETE BLOCK AND SLAB MAKING MACHINE.

compressed, and on the roller passing the end of the arm it is released, and therefore expands, pulling down the arm and causing the tamper (*f*) to strike the underside of the rack (*r*) which rests on the rocker (*s*). Arms (*e*) are weighted to counterbalance the tampers. Pedestal brackets (*o*) are placed at suitable distances longitudinally—for any length of rack to support the channel girders (*p*) to which trunnions (*t*) are attached. The rocker (*s*) rests on trunnions and the coil spring or rubber block (*x*) accelerates and controls the movement of them.

Fig. 3 shows end view of the traveller (*z*) on rail track (*q*) with tension pulley, bracket, and balance weight. The rails (*r*), resting on rocker (*s*), form a rack on which to place moulds containing concrete for consolidation.

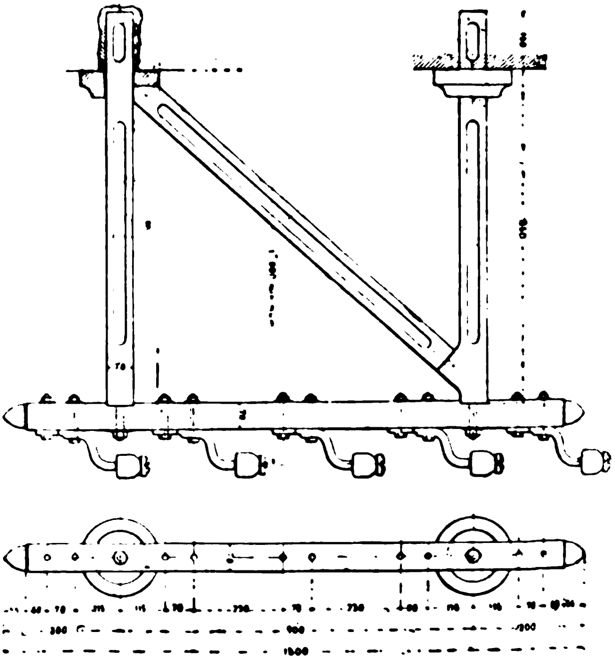
Figs. 5, 6 and 7 show a small machine with table 8 ft. by 4 ft., on which self-contained moulds may be placed for consolidation, and then removed for others to take their places. When it is desired to use the rack only, the table can be lifted off. This machine appears to us to be specially suited for the builder's yard, or the manufacturer of artificial stone.

ZIG-ZAG BRACKETS FOR PIPES AND WIRES.

According to M. J. Desaulty, of St. Quentin, brackets made of a rich cement-sand mixture have several advantages over other means of supporting pipes, wires, etc. They are both cheaper and much more durable than iron brackets, and involve no cost for painting or upkeep.

The type of bracket recommended (see illustration) is designed to carry five telephone and lighting cables, viz., 3 cables each 2½ in. diameter, 1 cable of 1½ in. diameter, and 1 cable of 1 in. diameter, with a space of 133 ft. between each bracket. The construction of the bracket is readily seen in the illustration, and a calculation of the bending moment and other static properties shows that this design combines a maximum strength and carrying power with a minimum of weight.

The mixture recommended consists of equal quantities of Portland cement and sand. The reinforcement of each of the two horizontal bars consists of 4 rods, each 0·33 in. diameter, tied at intervals of 4 in. with iron wire 0·08 in. diameter. The reinforcement of the inclined bar consists of 3 similar rods tied in a similar manner.—*Le Constructeur de Ciment Armé.*



[Measurements in Millimetres].
Detail of Concrete Bracket.

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ½ in. mesh	,,	15	0
Best British Portland Cement	per ton 58*s.* to	63	0
"Ferrocrete" Portland Cement	10*s.* per ton extra		

BOARDING FOR SHUTTERING—

		Sawn.		Wrot.	
		s.	d.	s.	d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in.	from £24 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £25 ,, ,,

MILD STEEL RODS FOR REINFORCEMENT—

		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
9/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(*Based on Contracts up to £2,000.*)

PORTLAND CEMENT CONCRETE WELL MIXED WITH CLEAN WATER IN PROPORTION 1 : 2 : 4—

				s.	d.
Do.	do.	in foundation	per yard cube	45	0
Do.	do.	in columns	,, ,,	50	0
Do.	do.	in beams	,, ,,	50	0
Do.	do.	in floor slabs 4 in. thick	per yard super	5	3
Do.	do.	in floor slabs 6 in. thick	,, ,,	7	9
Do.	do.	in floor slabs 9 in. thick	,, ,,	11	3
Do.	do.	in walls 6 in. thick	,, ,,	8	0

(*Add for hoisting* 3*s.* 6*d. per yard cube above ground-floor level.*)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	25	0
,, 9/16 in. to ½ in.	,,	24	0
,, ⅝ in. to 2⅞ in.	,,	23	0

SHUTTERING—

		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	54	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	45	0
Do. do. in small quantities	per ft. super	0	9
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	10
Do. do. as last, in narrow widths	,, ,,	1	0
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	0
Do. do. as last, in narrow widths	,, ,,	1	2
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

WAGES.—The rates of wages on which the above prices are based are those current in the London area, namely:—Carpenters and joiners, 1/7½ per hour; Labourers on building works, 1/2¾; Operators on concrete mixers and hoists, 1/3¾.

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

MEMORANDA.

PROSPECTIVE NEW CONCRETE WORK.

ABERDEEN.—*Harbour Works.*—The Aberdeen Harbour Commissioners have applied to the Unemployment Grants Committee for grants in aid of harbour improvement works.

ABERSYCHAN.—*Bridges.*—The U.D.C. proposes erecting two new reinforced concrete bridges at an early date.

ALBURY.—*Bridge.*—The Albury Council will contribute half the cost of a new overhead bridge at Albury railway station, estimated to cost £7,500.

ARDWORTH.—*Houses.*—The U.D.C. proposes to erect ninety-four houses in Sidcop Lane.

ASTWOOD BANK.—*Sewage Works.*—The Feckenham R.D.C. will apply to the Unemployment Grants Committee for a grant towards the cost of improving the sewage works at Astwood Bank, estimated to cost £3,700.

BANSTEAD.—*Houses.*—A company has bought 1,320 acres of " Nork Park " with the intention of developing the property; it is stated that 1,000 houses are to be erected.

BARROWFORD.—*River Wall.*—The U.D.C. is to spend £700 on the continuation of the concrete river wall.

BEBINGTON.—*Houses.*—The Bebington and Bromborough U.D.C. has given instructions to the Surveyor to prepare specifications and quantities for the erection of 100 houses.

BINGLEY. — *Houses.* — The U.D.C. has passed plans for the erection of thirty-two houses.

BLYTH AND CUCKNEY.—*Sewage Works.*—The R.D.C. has agreed to a scheme for sewerage and sewage disposal works at Haworth, at an estimated cost of £21,350.

BO'NESS.—*Houses.*—The Kinneil Coal Company, of Bo'ness, proposes to build 100 houses for its employees at a cost of £30,000.

BOOTHFERRY.—*Bridge.*—The West Riding C.C. proposes constructing a bridge over the Ouse at Boothferry, estimated to cost £100,000.

BOURNEMOUTH.—*Bridge.*—The Corporation will apply to the M.H. for sanction to borrow £1,000 for the construction of a bridge in the Bournemouth Chine Gardens.

BRADFORD.—*Houses.*—The Corporation will consider a proposal of the Health Committee for the clearance of the slum area in White Abbey, and a scheme for re-housing the persons in the area. The total cost of the scheme will amount to £240,000.

BRIDLINGTON.—*Road.*—The T.C. is considering the construction of a new road on the north of the town.

BURNHOPE. — *Roads.* — The Lanchester R.D.C. has applied to the Unemployed Grants Committee for grants in connection with the proposed new roads from Burnhope to Burnhope flats, Hedley Hill to Moorcock, and from Cutt Throat Lane to Hamsterley.

CHORLEY.—*Sewage Works.*—The T.C. has applied to the M.H. for sanction to borrow £50,000 for alterations and additions to the Common Banks sewage disposal works.

CIRENCESTER.—*Road.*—The U.D.C. has applied to the C.C. for support in making a new road from Grove Lane to Gloucester Road, estimated to cost £9,000.

CLYDEBANK. — *Roads.* — The T.C. has applied to the Government for grants in respect of four new road schemes, estimated to cost £140,000.

DARLINGTON.—*Houses.*—The T.C. will apply to the M.H. for approval to erect 604 houses.

DOVER.—*Harbour.*—The Harbour Board has approached the Trades Facilities Committee for a loan of £1,250,000 for the construction of a new dock of twenty-five acres.

DOVER.—*Harbour Works.*—The Harbour Board has taken over the naval harbour, and will have 700 acres of water enclosed to accommodate oil tanks and provide a commercial harbour.

DUMBARTONSHIRE.—*Houses.*—The Eastern District Committee of the C.C. has decided to erect 136 houses.

DUMFRIES.—*Houses.*—The T.C. is building forty houses at Creswell, at a total cost of £14,600.

EASTBOURNE.—*Sports Arena.*—A scheme is in hand for the construction of a large sports arena. A site at Whitbread Hollow has been suggested, as it forms a natural amphitheatre which would accommodate 250,000 people. The cost would be between £150,000 and £200,000.

FALMOUTH.—*Sewage Works.*—The Corporation has applied to the M.H. for sanction to borrow 65 per cent. of the estimated cost of their sewage disposal scheme.

FELTHAM.—*Houses.*—A scheme is in hand for the purpose of creating a garden suburb, to embrace 1,800 houses, thirty shops, public buildings, and factory accommodation at Feltham.

GLASGOW.—*Houses.*—The Housing Committee of the Corporation has recommended the purchasing of 195 acres, on which to erect some 2,000 houses.

GOOLE.—*Road.*—The U.D.C. is considering a scheme for the construction of a new road from Fountayne Street to Mount Pleasant.

HAMPSHIRE.—*Bridge.*—The C.C. is considering the construction of a bridge at Bursledon, estimated to cost £35,000.

Hinckley.—*Roads, etc.*—The U.D.C. has received sanction to borrow £5,412 for the construction of roads, etc., on the Rugby Road building estate.

Horndean.—*Reservoir.*—The R.D.C. has submitted plans of reinforced concrete reservoirs to the M.H. for approval.

Hornsea.—*Sea Defence.*—The U.D.C. has taken up a loan of £35,756 for sea defence work.

Huddersfield. — *Sewage Works.* — The T.C. has applied to the Government for a grant towards the cost of the proposed extension of the sewage works at Deighton, estimated to cost £18,600.

Ibstock.—*Sewage Works.*—The Market Bosworth R.D.C. has applied to the M.H. for sanction to borrow £6,670 for sewage disposal works.

Kettering.—*Houses.*—The U.D.C. proposes to build 300 houses during the next two years.

Lancing.—*Groynes.*—The Shoreham and Lancing Sea Defence Commissioners will borrow £4,000 for the erection of four new groynes at the eastern end of the foreshore.

Leamington.—*Houses.*—The T.C. has a scheme in hand for the erection of 260 houses.

Leeds.—*Road.*—The T.C. has received sanction from the M.H. to borrow £27,957 in connection with the construction of a new road from Headingley to Moortown.

Leek.—*Houses.*—The U.D.C. has applied to the M.H. for sanction to borrow £14,500 for the erection of thirty-six houses.

Llanelly.—*Houses.*—The G.W.R. Co. will provide financial assistance for the erection of 200 houses for local employees.

Manchester.—*Houses.*—The Corporation Housing Committee has approved of the lay-out of the Burnage estate, on which it is proposed to build 788 houses.

Manchester.—*Roads.*—The Corporation has submitted schemes for four new roads, estimated to cost £200,000, to the Council Finance Committee and the M.H.

Margate.—*Road.*—The Corporation proposes to construct a new road in front of Van den Burgh building estate.

Montrose.—*Bridge.*—The Bridge Joint Committee has submitted plans to the T.C. for a new bridge over the river South Esk, estimated to cost £80,000.

Norton.—*Houses.*—The Stockton T.C. will build 100 houses on the Blue Hall estate.

Ollerton, Notts.—*Houses.*—The Butterley Building Co. is to build a village, comprising 1,000 houses.

Paisley.—*Houses.*—The T.C. has decided to build 100 houses at Whitehaugh.

Royton.—*Sewage Works.*—The U.D.C. will apply to the M.H. for sanction to borrow £24,053 in connection with the extension of the sewage disposal works.

Rutherglen.—*Houses.*—The T.C. will build fifty-eight more houses on the Gallowflat site.

Sandbach.—*Houses.*—Messrs. Brunner, Mond and Co. propose to build houses at Sandbach for their employees.

Stafford. — *Houses.* — The T.C. has received the approval of the M.H. for assistance in the erection of forty houses.

Stalybridge.—*Road.*—The T.C. will apply to the M.H. for sanction to borrow £4,650 for the construction of a new road from Harrison Street to Cambridge Street and Church Walk.

Staveley. — *Houses.* — The Industrial Housing Association is considering a scheme for the construction of 1,000 houses on the Hollingwood site.

Sunderland.—*Road.*—The M.T. has advanced a grant of £14,996 towards the construction of a new section of a ring road.

Swansea.—*Bridge.*—The G.W.R. Co. are preparing plans for a bridge over the level crossing at the King's Dock.

Tendring (Essex).—*Sewage Works.*—The R.D.C. are seeking sanction to borrow £5,700 for sewage disposal works.

Thorpe-le-Soken.—*Sewage Works.*—The R.D.C. has applied to the M.H. for sanction to borrow £5,700 for sewage disposal works.

Thurnscoe.—*Houses.*—Plans have been drawn for the erection of 400 houses.

Turton.—*Sewage Works.*—The U.D.C. has applied to the M.H. for sanction to borrow £9,000 for extensions and alterations at the sewage disposal works at Eagley.

Weston-super-Mare.—*Sea Wall.*—The U.D.C. is investigating the question of enclosing Glentworth Bay by the erection of a new sea wall.

Whitwood.—*Road.*—The U.D.C. proposes constructing a new street from Teall Street to Station Road.

Wortley.—*Road.*—The R.D.C. proposes constructing a new road from the " Crown " Inn to Hangmanstone Bar, estimated to cost £60,000.

Yeovil.—*Houses.*—The Corporation has agreed to erect another 160 houses.

TENDERS ACCEPTED.

Adwick-le-Street. — *Reservoir.* — The U.D.C. has accepted the tender of Messrs. Swift Bros. & Haslam, Ltd., Doncaster, at £3,381 5*s.* 6*d.*, for the construction of a concrete reservoir to hold 500,000 gallons.

Cullompton (Devon). — *Bridge.* — The tender of Messrs. Fothergill Bros., Exeter, at £2,700 for the erection of a ferro-concrete bridge near Cullompton, has been recommended for acceptance.

Glasgow.—*Bridge.*—The tender of Messrs. Melville, Dundas and Whitson, Glasgow, at about £100,000, for the construction of a concrete bridge over the Clyde at Oswald

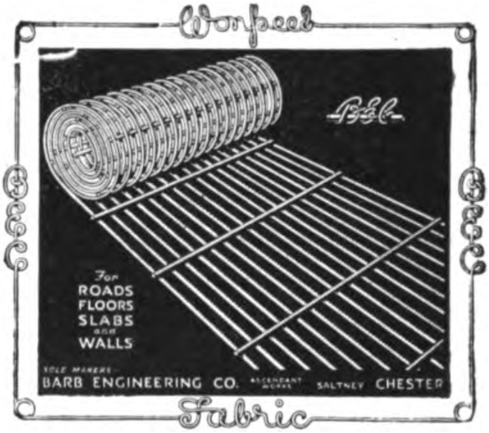
Street, has been recommended for acceptance. [An illustration of this bridge was given in our July number.]

GLASGOW.—*Works.*—The Corporation has accepted the tender of Messrs. W. Taylor & Sons, Ltd., at £2,145 11*s.* 11*d.*, for reinforced concrete work at Dalmarnock power station.

MELTON MOWBRAY.—*Waterworks.*—The U.D.C. has accepted the tender of The Monnoyer British Construction Co., London, at £11,499, for the construction of a concrete service reservoir with a capacity of 1,000,000 gallons, pump house, tank, and high water level tower, etc.

WETHERBY.—*Reservoir.*—The Wetherby and District Waterworks Company has accepted the tender of Mr. P. Charnock, at £1,328, for the construction of a concrete storage reservoir, 100 ft. by 55 ft. by 10 ft. deep.

WHITEHAVEN.—*Dam.*—The T.C. has accepted the tender of Messrs. J. Leslie & Sons, at £1,500, for the construction of a dam in the South Tidal Harbour.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CONCRETE AND CONSTRUCTIONAL ENGINEERING

NOVEMBER 1923 VOL. XVIII. No. 11

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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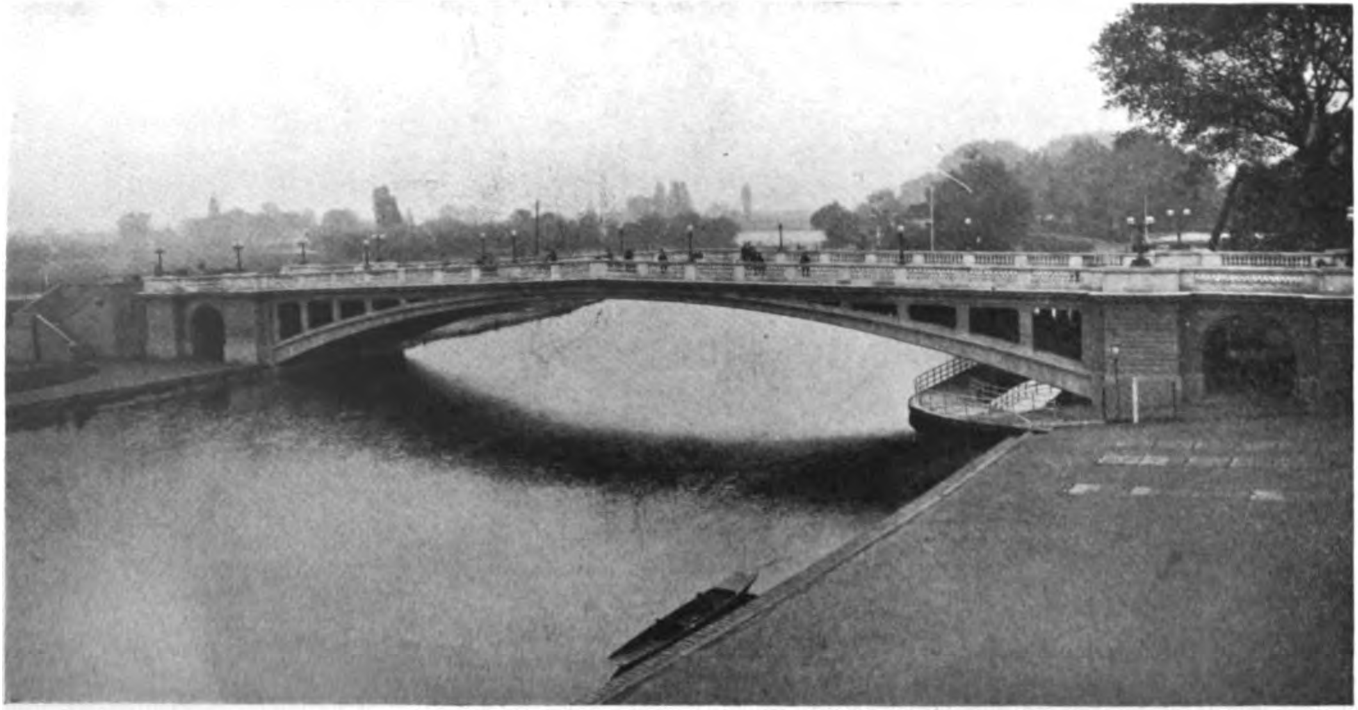

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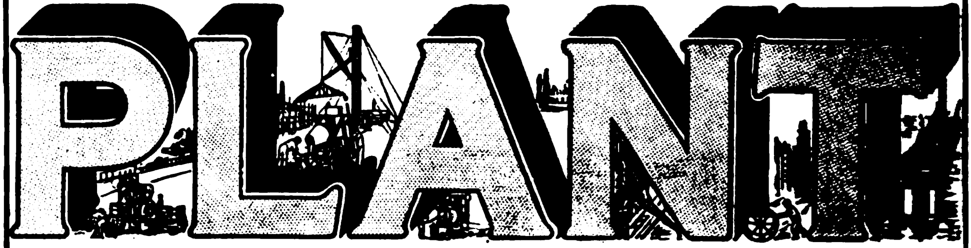

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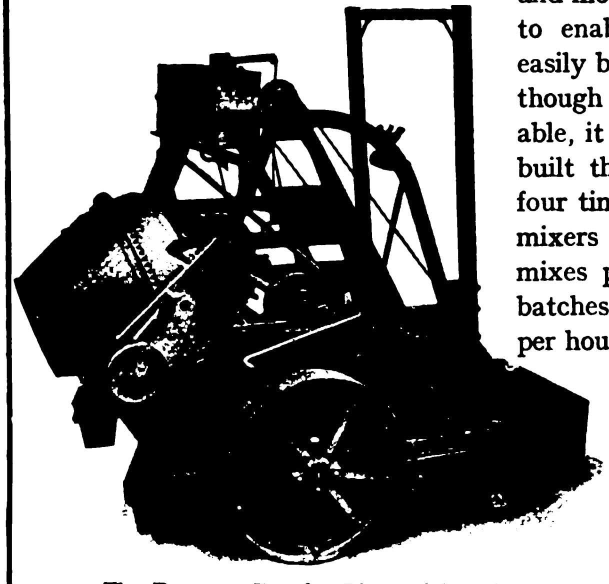

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CONCRETE
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CONSTRUCTIONAL ENGINEERING

Volume XVIII. No. 11. LONDON, NOVEMBER, 1923.

EDITORIAL NOTES.

CONCRETE RESEARCH.

ON making a general survey of the work of the past few years the advocates of concrete can find much cause for satisfaction. It is true there are many uses to which concrete may still be put and many improvements possible to present-day concrete designs and workmanship, but the progress that has been made in the development of concrete throughout the world is sufficient to satisfy those who know the difficulties to be overcome in changing the constructional methods sanctioned by several decades of experience. But the same general survey cannot fail to impress the open-minded observer with the large scope there is for the improvement of the material concrete itself, and it is well to recapitulate from time to time what requires to be done to advance still further the claims of concrete to be a pre-eminent constructional material.

Perhaps the most noticeable direction in which improvement is desired is in connection with the tendency of concrete to contract. In order to produce a workable concrete it is necessary to use more water than is required for the setting of the cement, and in consequence of the removal of the excess water by evaporation there is a tendency for concrete to contract. Under certain conditions the contraction leads to cracking, and it has to be admitted that the surface of concrete is frequently spoilt by this means. In avoiding the appearance of cracks—even no more than " hair cracks "—the concreter has to abandon the production of a smooth polished surface which would, if perfect, greatly enhance the beauty of concrete and bring it into line with polished natural stone. Whether the remedy for such contraction lies with the cement manufacturer or the concreter is a question unanswered. The specifications for cement do not deal with contraction, and it does not seem to be known whether some cements are more liable to contraction than others or whether all are alike in this respect. For his part, the concreter knows that if he attempts to produce a very smooth surface to his concrete he is running the risk of getting cracks ; but so far as we are aware it has not been agreed as to when, in relation to the time of setting of the concrete, the surface should be finally trowelled in order to produce the smoothest result consistent with absence of contraction cracks, or whether it is possible by any treatment to combine a highly-polished surface with freedom from cracks or hair lines. It is obvious that the contractive tendencies of concrete do not affect the work for which it is now adopted, but the removal of such tendencies would extend the uses of concrete.

Another direction in which more knowledge is needed is in connection with the resistance of concrete to acids, oils, and other industrial liquids. With many materials, such as mineral oils, unprotected concrete tanks are quite successful, and the advantages of concrete for factory and storage plants are so marked that several industries would welcome the introduction of concrete which would resist the action of acids and certain other substances.

A subject that is frequently discussed among concreters may perhaps be described as the "individuality" of cements. In spite of standard specifications, preferences for certain brands of cement are still so strong that it is unwise to dismiss them as "prejudice" and to suggest that all cements complying with the Standard Specification are alike. This again is a matter for inquiry, and if possible the root causes of the popularity of certain brands should be discovered and tests for the corresponding qualities embodied in the Standard Specification for Cement.

This is not the place to present a comprehensive list of improvements and investigations that are needed to advance the development of concrete, but a score of unanswered questions will crowd into the minds of all who have been much occupied with cement. What constitutes a dirty aggregate? What proportion of sulphur is permissible in a slag to be embodied in concrete? When a concrete fails to harden, how can the cause of failure be ascertained? These are some of the questions still requiring investigation and research before they can receive a satisfactory answer.

It is not at all certain that in England the problems of concrete are receiving the attention they need. So far as the principal ingredient of concrete, namely, the cement, is concerned, it is to be expected that the research organisation of the cement manufacturers will attack the problems connected with that material, and possibly, when further knowledge has been obtained, the British Standard Specification may embody tests of the properties of cement which are now ignored by specification tests. But concrete aggregates and concrete manipulation are in another category, and should be the concern of another body which at the moment does not appear to exist.

CONCRETE TUBES.

THE article in this issue from the pen of Dr. Emperger is one which should interest a great number of our readers. Not only is the subject in itself an interesting one, having regard to the increasing use that is being made of concrete pressure pipes, both plain and reinforced—pipe manufacturing being quite an industry of its own—but special interest attaches to the fact that the subject is being treated in all its bearings by such a well-known authority, who has contributed many an excellent work on the theory and practice of reinforced concrete. One of the many points of general importance that is dealt with in the article is the question of the value of the modular ratio *m*, which enters into practically every form of calculation of reinforced concrete structures and is therefore a most important factor to the designer, and one on the determination of which a good deal of research work has been spent. It is generally assumed, for ordinary structural calculations, that the value of m is 15, that it is constant whatever the stresses in the concrete may be, and that it is identical for compression and for tension. Dr. Emperger deals with this incorrect assumption very exhaustively and shows that the value 15 is only one of the many m may have, while it varies considerably with the stresses; and that, further, it is far from being the same for compression and for tension, this fact being already recognised by the Austrian regulations which place the values for compression and for tension at 15 and 37. Our own L.C.C. regulations take cognisance of the quality of the concrete, or rather of the allowable working stress determined by this quality, in attributing different values to m; but whereas the L.C.C. regulations prescribe a smaller m with increasing stress (from 15 for $c = 600$ lbs. per sq. in. to 9 for $c = 1000$ lbs. per sq. in.) it would appear from Dr. Emperger's tests that the variation of m is in the other direction. As will appear from the test results given, the value of m may be, in certain circumstances, as high as 60.

To those specially interested in concrete pipe and allied work, such as reservoirs, stand-pipes, etc., the new system of pressure pipes described as the "Emperger-Umlauf" should appeal as a novel and original method of construction. This invention solves one of the fundamental difficulties inherent to the design of reinforced concrete pressure pipes; in the ordinary way it is impossible, in a tension structure, to develop the full tensile strength of the steel reinforcement without stressing the concrete in tension too highly, more particularly so in the case of high grade steel; this is overcome in the new system by imparting to the concrete an initial compression stress which must be overcome and neutralised by the internal pressure before the concrete begins to feel any tension at all. The method of producing this initial compression by laying the reinforcement in the form of a tension ring outside the concrete pipe is not a new idea in itself, although it is probably the first time it has been applied to reinforced concrete; Louis Brennan, the well-known inventor, made use of exactly the same principle and designed a somewhat similar device for the flywheels of his gyroscopes. In many respects the problem of the pressure pipe is analogous with that of the flywheel rim where the centrifugal force corresponds to the internal pressure in the pipe. Owing to the extremely high revolution speed employed in the gyroscopes it was found difficult to prevent the flywheels from bursting, if made from ordinary materials; Brennan therefore shrunk a ring of special high tensile steel round the flywheel, the initial shrinkage tension in the ring producing a compression stress in the flywheel which acts as a set-off against the tension produced by the centrifugal force. Applying this principle to reinforced concrete pipe construction, it will be seen that by varying the initial tension in the reinforcement it is within the power of the designer to keep the tension stresses in the concrete within any desired limits while yet utilising to the full the tensile strength of the steel reinforcement; it is even possible to prevent any tension stresses at all from developing in the concrete and, in fact, make cracked pipes watertight by inducing compression stresses that close up the cracks.

The first part of the article appearing in this issue deals mostly with the theoretical aspect of the various forms of pipe construction; the second part is devoted to the description of the tests carried out with pipes built on the "Emperger-Umlauf" system.

REINFORCED CONCRETE REGULATIONS.

It is on the whole a good thing for the development and progress of reinforced concrete as a method of construction that the erection of buildings in that material should be governed by regulations which ensure good design, materials, and workmanship; if the municipal authorities had no authority to enforce compliance with such regulations buildings would no doubt be erected by unqualified people and accidents would be sure to occur which would reflect badly upon the whole industry.

At present the London County Council Regulations have effect far beyond the London boundary because many other municipalities adopt them as their standard. These regulations are now six years old, and although they are excellent in parts they are in many respects too involved and in some respects so onerous that they add unnecessarily to the cost of buildings. In a few respects, on the other hand, many engineers believe that they might be made more stringent. In suggesting, therefore, that the time has now arrived for a complete revision of the Regulations in the light of six years' experience in their use, we are not influenced so much by the irresponsible people who want to be allowed to do as they like as by the current opinion among leading structural engineers who are anxious that London should have a set of regulations which would ensure good construction while giving scope for originality and development. The danger in making regulations so stringent that they effectively prevent bad design on the part of ignorant designers (or of designers who ought to know better) is that in the process the regulations may be made so fixed that they prevent the best designers from giving full scope to their ingenuity and resource. The problem, therefore, confronting those who are responsible for regulations is to steer between the vagueness which does not ensure good construction and the inelastic rigidity which kills enterprise.

While our thoughts are occupied with the subject of by-laws and regulations we must remind our readers of the unwise procedure of having different regulations for different towns; building regulations should be a national matter and should not be different for different neighbouring localities. The present system under which every municipal authority is a law unto itself leads to a great deal of unnecessary trouble and annoyance to architects and engineers, and sometimes the absurd condition arises that a thing which is permitted on one side of the road is forbidden on the other. If the difficulty is raised as to which municipal authority is to lead with regulations so that others may follow, we suggest that no municipal authority should draft regulations. They should be drafted by a committee representative of all the professional and other interests concerned.

REINFORCED CONCRETE BRIDGES.

The decision of the Government to allocate a large sum of money for road and bridge construction during the coming winter will fulfil the dual purpose of securing many much-needed public improvements and alleviating distress by providing employment on a large scale. In view of this decision we give in this issue illustrations of recently erected bridges of the type which it is to be anticipated will be built in considerable numbers in all parts of the country.

CONCRETE TUBES FOR INTERNAL PRESSURE.

By DR. F. EMPERGER.

THE only factor that is of importance in connection with the strength of concrete tubes subject to internal pressure is the ability of the material to withstand tension. In spite of the fact that the tensile strength of concrete prepared with high-class cement is comparatively high, it cannot be said that concrete tubes for this purpose have been used extensively, the principal reason being that the tensile strength of concrete is known to be somewhat uncertain and erratic, and depending to a very great extent upon the method of manufacture, and that therefore the factor of safety would need to be very high in order to secure reliable working conditions. As soon, however, as the concrete in the tube is reinforced with steel the latter will prevent the simple bursting of the tube which takes place somewhat on the lines indicated in *Fig.* 1, where a plain concrete tube is shown after having been tested to destruction. The function of the reinforcing steel in concrete tubes is, however, a problem on which a good many erroneous ideas exist.

One of the fundamental rules applying to ordinary reinforced concrete work recommends that no account should be taken of the tensile strength of concrete in calculating any structural design; this rule, however, does not hold good in the case of concrete tubes dealt with here. Also it must be remembered that the factor of safety for concrete tubes cannot be compared with a corresponding factor against cracking of the concrete in ordinary structural design, because in the latter case the appearance of cracks need have no relation to the ultimate strength, and the safety provided against cracks has for its purpose only the prevention of these within the limits of the working load. In this case it is therefore permissible to use comparatively high working stresses for concrete in tension, up to about two-thirds of the ultimate stress; or, in other words, roughly speaking, a factor of safety of 1½ against crack formation.

The cracks in concrete tubes must, however, be considered from quite another point of view. When cracks commence to form they put the tube out of commission, the appearance of cracks being for all practical purposes equivalent to destruction, and for this reason this particular application of reinforced concrete is in a class by itself, it being no longer safe to consider the reinforcement as carrying all the tension. The steel in concrete tubes only serves to increase the tensile strength of the concrete, but the exact manner in which this increase of strength is brought about is not by any means fully cleared up, at least not experimentally.

These different view points must be borne in mind in trying to solve the problem of determining the ultimate strength of high-pressure concrete tubes, and must influence the choice of the proper factor of safety. For this particular branch of reinforced concrete work it is necessary to use a factor that appears high, at least in comparison with ordinary structural work. Special difficulties are encountered in the design of a simple and efficient joint, a question which will be dealt with at a later stage, and in the case of exposed tubes there is in addition the problem of temperature changes requiring the formation of special expansion joints, in regard to which latter fairly exhaustive experiments have been carried out. A full report of the latter is given later, together with a description of the "Umlauf" joint, which solves all the technical difficulties; it is sufficient, at this stage, to mention that the joints can be satisfactorily designed, and that this detail has been tested and found efficient, so that this question may be left out of consideration when endeavouring to formulate the statical calculation for concrete pressure tubes, having regard to what has been said above in connection with the tensile strength of concrete.

I.—PLAIN CONCRETE.

In calculating the dimensions of a tube having an internal diameter of $2r$ in., for a pressure of p lbs. per sq. in., the thickness of the walls must be such as to enable them to take a total tension P equal to rp lbs. per in. length of tube. If c is the allowable working

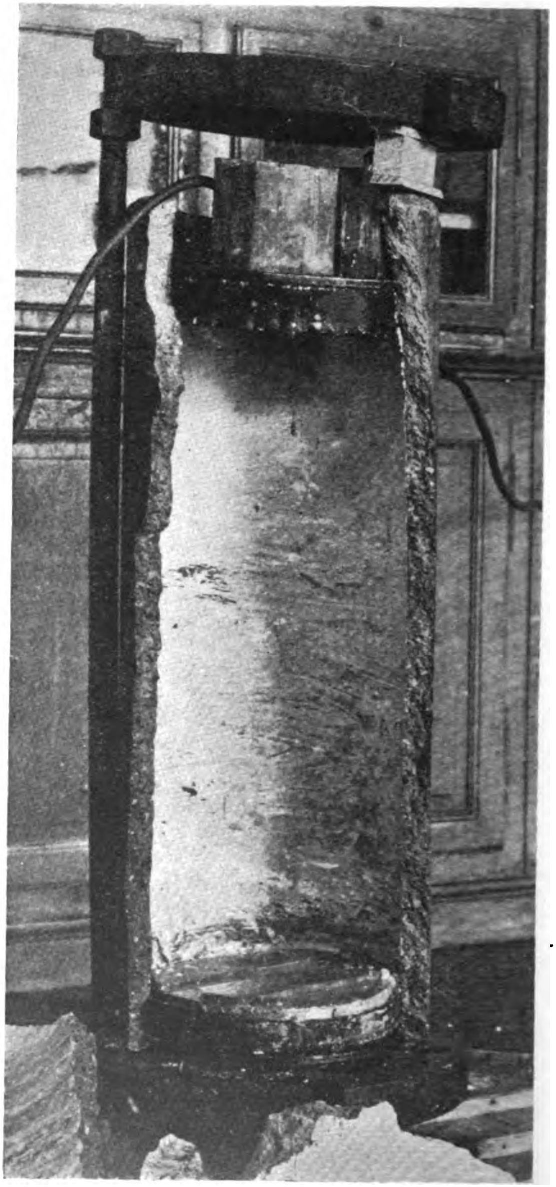

FIG. 1.

stress of concrete in tension, then the thickness t in inches of the wall is found from

$$P = t \times c = r \times p \quad . \quad . \quad (1)$$

or for an ultimate strength of 5 times the working strength

$$5P = t \times c_u = r \times p'$$

The factor of safety of 5 suggested here would, however, hardly appear sufficient. In order to arrive at the ultimate tensile stress c_u of the concrete used in the experimental tubes described in the following, two tests to destruction were carried out on plain concrete tubes, the so-called Kielberg tubes manufactured by the firm of Steinag in Vienna (*Fig.* 1) with the result that the two tubes withstood a pressure of 4 and 6 atmospheres respectively, or an average of 5 atmospheres. If the factor of safety is chosen as 5, the tubes in question would therefore appear to be suitable for a working load of 0·8 atmosphere, as the basis must be taken to be the minimum, not the average of the two tests. The concrete in question, owing to the special method of manufacturing the tubes, was cast under compression, and the ultimate strength of the concrete found during the tests corresponded to 355 lbs. per sq. in., which would give a permissible working stress of 70 lbs. per sq. in. With concrete cast in the ordinary way a working stress of about 40 lbs. per sq. in. would be permissible.

II.—REINFORCED CONCRETE.

In reinforced concrete tubes with a steel reinforcement of an area A sq. in. the working load or total tension in the wall for a length of 1 in. of the tube is

$$P = r \times p = c(t + mA) \quad . \quad (2)$$

or for an ultimate load of $2\frac{1}{2}$ times the working load

$$2{\cdot}5\,P = r \times p' = c_u(t + mA)$$

The ultimate strength of the concrete c_u may be taken as 200 lbs. per sq. in. for ordinary concrete, and 355 lbs. per sq. in. for compressed concrete such as is used in the Kielberg tubes. A still further increase in the ultimate strength would appear permissible in the case of concrete tubes where the material is placed by centrifugal force or shot by compressed air ; no tests are, however, available for concrete produced in this manner. Only very few scientifically conducted tests on reinforced concrete tubes have been carried out ; the series of tests mentioned below was undertaken at Stuttgart. The tabulated results of the tests were as follows :

Tube No.	1	2	3	4
Working pressure in lbs. per sq. in. . .	70	70	140	140
Internal diameter in inches	35	35	28	28
Thickness of walls in inches	7	7	10	10
Area per ft. of reinforcement in sq. in. .	1·82	1·82	2·75	2·75
Pressure in lbs. per sq. in. at first crack .	140	140	256	228
,, ,, ,, at destruction .	140	140	280	256

An ultimate pressure somewhat higher than that at which the first crack appears is, of course, only possible if the test pump can supply water pressure at a greater rate than that lost through the cracks, and the last figure gives the pressure at which the tensile strength of the concrete was actually exhausted. The ultimate pressure for the two sets of tubes amounted to 140 lbs. and 268 lbs. per sq. in. respectively, the latter figure being the average of 280 and 256—in other words, a factor of safety of only 2 over the working pressures.

If the reinforcement is left out of consideration, the above figures give the following equations for calculating the ultimate tensile stress of the concrete,

$$r \times p' = t \times c_u$$

$$\left.\begin{aligned} \text{or} \quad & 17{\cdot}5 \times 140 = 7 \times c_u\,; \\ & c_u = 350 \text{ lbs. per sq. in.} \\ \text{and} \quad & 14 \times 268 = 10 \times c_u\,; \\ & c_u = 375 \text{ lbs. per sq. in.} \end{aligned}\right\} \quad . \quad (3)$$

The average calculated strength is, according to these equations, 367 lbs. per sq. in. and the uniform results testify to the careful method of manufacturing the tubes. The influence of the steel reinforcement on the calculated stress depends on the choice of the factor m in equations (2). The percentage of steel in the above tubes is roughly 2¼, and if *m*, for concrete in tension, is taken as 15, 30, or 45 respectively, the effect of the 2¼ per cent. of steel is 34 per cent., 67 per cent., and 100 per cent. respectively, and in the three cases the actual ultimate tensile strength of the concrete would therefore be 270 lbs., 228 lbs., and 185 lbs. per sq. in. respectively.

There is not sufficient difference between these figures to enable any conclusions to be drawn from the tests as to the best choice of the factor *m*, but from the above it will be seen that, besides *m*, the other very important factor is the ultimate tensile strength of concrete c_u, which must be settled experimentally or assumed before the ultimate strength of the tube can be calculated. For this to be done correctly, the ultimate tensile strength of the concrete in question must therefore be found by actual tests. In the report of the above trials the only tests on the concrete from which the tubes were made were the usual crushing tests on cubes which gave an ultimate crushing strength of 3,250 lbs. per sq. in.

The actual report reads as follows: "Direct tension tests were unfortunately not carried out, it being considered sufficient to assume an ultimate tensile strength of 215–255 lbs. per sq. in., having regard to the results of the crushing tests, and the usual ratio between the crushing and the tensile strength."

This assumption corresponds to a ratio of about 15 between the two latter. It is, however, not correct to assume that the strength of the tubes can be deduced from the average value of the ultimate tensile strength (235 lbs. per sq. in.); it is the weakest spot in the whole tube that is responsible for the appearance of the cracks. To be on the safe side a reduction of 20 per cent. of the average value should be made in order to obtain the value to be used in the calculation; this would correspond to a value of 235 lbs. less 20 per cent., or approximately 185 lbs. per sq. in. for the above trials. This would agree with the value of $m = 45$ found above on this assumption. An exact calculation based on $m = 15$ gives still higher tensile stresses than those first arrived at above (up to about 450 lbs. per sq. in.), and the same exact calculation would therefore require a far higher value of *m* than 45 to make the actual tensile strength agree with the calculations. The exact calculation is here omitted, as there does not appear to be much purpose in finding the stress in the extreme fibres when not even the basis value of the tensile strength of the concrete has been ascertained experimentally.

As an example of obtaining an estimate of the maximum limit for the use of reinforced concrete pressure tubes of an internal diameter of 63 in., manufactured from concrete having an ultimate tensile strength of, say, 215 lbs. per sq. in., a reinforcement of, say, 2 per cent., and with a factor of safety of 2·5, the following equations may be drawn up

from (2) $2{\cdot}5\,P = r \times p' = c_u(t + mA)$,

or $$31{\cdot}5\,p' = 215(t + 0{\cdot}02mt) \quad . \quad (4)$$

$$p = \frac{p'}{2{\cdot}5} = \frac{215}{2{\cdot}5}(1 + 0{\cdot}02m)\frac{t}{31{\cdot}5} \quad (5)$$

For a thickness of the walls of 10 in., the allowable working pressure *p* is found

for $m =$	20	30	45
as $p =$	38·2	44·0	52·0 lbs. per sq. in.
or	2·6 atm.	3 atm.	3·6 atm.

For reasons shown later the most suitable choice of *m* seems to be in the neighbourhood of 45, and in the following this value will be considered as a basis for further calculations; it may be mentioned that the highest working pressure generally in use for reinforced concrete tubes without special sheet steel lining is about 4 atmospheres (57 lbs. per sq. in.). The correctness of the assumption made with regard to the value of *m* as 45 can be fairly proved by means of tests on reinforced concrete beams for the appearance of cracks in the tension zone, and it is a question that is intimately dependent upon the ductility of concrete.

The latter property of the material has been exhaustively examined and tested by Considère and Mesnager in France, and by Schule, Kleinlogel and Bach in Germany, the results of the tests being reported upon by Graf in the "Handbuch für Eisenbeton," 3rd edition, Vol. I, pp. 85–103. It may be of interest to mention the conclusion arrived at from these tests, which is stated in the following terms : " Under the loads and stresses usually allowed under the various official regulations, the formation of cracks generally occurs."

It must, however, be remembered that in the case of reinforced concrete beams the shrinkage of the concrete in setting is a contributive factor of some importance when dealing with the question of cracks, and the fact that, notwithstanding, so many reinforced concrete beams under working conditions are free from cracks goes to show that careful design and workmanship can do much to prevent these.

The shrinkage of concrete does not, however, figure to any extent in the case of tubular work, and it is therefore possible to exploit the whole of the ductility of the material in pure tension. The ductility of reinforced concrete is stated by the German investigators to be 0·1–0·3 mm. per metre length (1¼ to 3½ thousands of an in. per ft. length), or practically the same as for plain concrete, while the French research gives results that are about ten times these values. These contradictory results may be explained by the method of conducting the tests, the German experts registering as " destruction " of the concrete the moment when water was percolating through the minute cracks ; the presence of steel reinforcement allows a distribution and subdivision of cracks along the whole length of the steel bars, and the concrete is thus enabled to continue its extension, without, however, any real increase in the tensile strength ; the same phenomena has been proved by similar tests on concrete in compression.

In considering the German tests from this point of view, it is permissible to increase the figures mentioned above to 0·5 mm. per metre length (⅔ thousandths of an in. per ft. length). Apart from this consideration the main difference between the French and the German results must be attributed to the extraordinary quality of the concrete under test, a quality which could never be equalled in actual practice. It must be borne in mind that, in placing the concrete material in reinforced concrete tube work, due consideration must be given to the fact that a fairly close network of steel bars on both sides of the concrete thickness will prevent the use of concrete of any but a fairly liquid consistency, and that moreover the ramming or tamping of the concrete for the same reason can never be carried out so thoroughly as in ordinary open work ; the tensile strength of concrete so placed cannot therefore be taken with safety as more than 215 lbs. per sq. in. But the corresponding extension of the steel embedded in the concrete is not equivalent to a steel stress of 15 times 215 lbs. per sq. in., or 3,250 lbs. per sq. in., at which stress the extension would be

$$\frac{3{,}250 \times 12}{30{,}000{,}000} \text{ in. per ft. length}$$

or 1¼ mille, at which stage the percolation of water would no doubt take place ; the extension of the concrete, which, however, is no longer elastic, is continued until a stress of at least 45 times 215 lbs. per sq. in., or 9,750 lbs. per sq. in., in the steel is reached, at which stress the corresponding elongation in the latter is

$$\frac{9{,}750 \times 12}{30{,}000{,}000} \text{ in. per ft. length}$$

or 3½ mille.

In this connection reference may be made to the article of the author in *Beton u. Eisen*, 1916, pp. 59–96, entitled "The Real Magnitude of the Inertia Moment in Reinforced Concrete Beams," where it is also shown that at first a small decrease in tensile strength takes place, and only at a much later stage does the tension of the concrete itself become exhausted. The variations of the value of *m* for concrete in tension are plotted, and the results show that, while the value of *m* for tension is the same as for compression at the commencement of the load application, when the stresses are small the former value increases hyperbolically with the stress. This is shown particularly clearly in the increase of *m* for tension from equality to the 5-fold value of *m* for compression, which for an *m* of 7 to 15 in compression would correspond to an *m* of 7 to 75 in tension.

The value of *m* for tension can also be calculated in the ordinary way by testing reinforced concrete beams up to the point where the first crack appears, and expressing in equation the fact that a moment before the crack appeared (*i.e.*, at the load and under the action of the bending moment obtaining just then) the beam is assisted in carrying its load by the concrete acting in tension, the concrete being in fact at that moment stressed to its ultimate tensile strength. This point, according to the report by Graf mentioned above, is reached together with the permissible stresses in the steel, viz. 16,000 lbs. per sq. in., neglecting the tensile strength of the concrete as is usually done in the ordinary calculation, the bending moment

$$B.M. = R_t = 16{,}000\,A\left(d - \frac{n}{3}\right) \quad (6)$$

At the moment of cracking the concrete is, however, taking tension up to its ultimate limit, and the depth of the neutral axis, when allowing for the concrete taking tension stresses, is found from

$$n = \frac{\frac{bd^2}{2} + mAd}{bd + mA} \quad . \quad . \quad (7)$$

where the simplification is introduced of making $d = d_t$; using the actual values of a beam under test, the value of m for tension was found as 57.

In this calculation the formulæ used are derived on the assumption that the modular ratio m is the same for compression and tension, is constant under varying stress, and is in numerical value 15. In the cases where these formulæ are generally used it is an assumption that does not essentially influence the calculation of beams or other structural members, as no account is taken of the ability of concrete to take tension. In calculating the tensile stresses the latter appear much too high, but this error may be compensated by assuming very high allowable stresses.

For the case under consideration this method of calculation is hardly exact enough. In order to approach closer to the actual facts, a different value for the modular ratios for compression and for tension must be used as basis, such as is provided for in the Austrian Regulations where the m for compression is prescribed as 15, whereas the corresponding figure for tension is 37·5. In the case of a structure essentially in tension, such as the one dealt with here, a more correct assumption would be to take $m_c = 7{\cdot}5$ and $m_t = 45$ for compression and tension respectively, giving a ratio of 1 to 6 between the two modular ratios. When the calculation is carried out on this latter assumption the following equation obtains * :—

$$\frac{bn^3}{3} = \frac{m_c}{m_t} b \frac{(d - n)^3}{3} + m_c A (d - n)^2 \quad (9)$$

This equation gives a value of the tensile stress in the concrete of 215 lbs. per sq. in., which makes the assumption of $m_t = 45$ agree with the fact that a crack is actually appearing in the concrete where the tension steel is placed.

In the following the value $m = 45$ will therefore be used as modular ratio for concrete in tension at the moment when cracks are beginning to appear, and as for the consequences of this alteration of m it is only necessary to mention that this assumption does not make a great deal of difference to the results as calculated from the ordinary formulæ, because although the m in the latter is taken too small, the allowable stresses make up for this by being taken too high. In the case of ordinary concrete of a tensile strength of 215 lbs. per sq. in., the allowable tension stress is taken as 85 lbs. per sq. in., and only in exceptional cases, where the concrete has a tensile strength of 350 lbs. per sq. in., the allowable tension stress may be taken as 140 lbs. per sq. in.

III.—REINFORCED CONCRETE WITH SHEET STEEL LINING.

In using an internal steel lining a wall is obtained which is independent of any cracks forming in the concrete, but where, on the other hand, the collaboration of the concrete as tension member is lost as soon as the ultimate tensile stress has been exceeded. If the tube is to have a factor of safety for working pressure of 1·5 against this contingency, then the allowable tension stress for concrete with an ultimate strength as in the two cases above, would be 140 lbs and 240 lbs. per sq. in. respectively, and the allowable load per inch run of tube

$$P = p \times t = c(t + mA). \quad . \quad (10)$$

or the ultimate load

$$1{\cdot}5P = p' \times t = c_u(t + mA) \,. \quad (11)$$

If now m is taken as 45, then steel stresses of 9,600 lbs. and 16,000 lbs. per sq. in. respectively are arrived at, according to the quality of concrete, at which stress the concrete ceases to play any

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* See *Handbuch für Eisenbeton*, 3rd edition, vol. 1, p. 619.

part in taking tension, because it commences cracking. If therefore the steel is required to be fully utilised it will be necessary to neglect the presence of the concrete for taking tension, but to observe that the limit of the tensile strength of the concrete is not yet reached when the steel reinforcement is stressed to its permissible load. The principle is the same as that followed in calculating a structure which is to be free from cracks under ordinary working loads, but where no help is expected from the co-operation of the concrete.

IV.—INITIALLY STRESSED REINFORCEMENT.

In the system known as the "Emperger-Umlauf" the finished concrete tube is encircled by reinforcement, either a continuous spirally-wound wire or separate rings, to which initial tension is imparted, so that the concrete tube, in reacting against this, obtains an initial compression. It therefore follows that the internal pressure to which the tube is subject must first of all neutralise this initial compression before the tensile strength of the concrete is called into play. Let the initial tensile stress of the steel reinforcement be s_o and the resulting compression stress in the concrete be c_o, then, if the thickness of the shell and the steel area per inch length of tube be t and A respectively

$$As_o = tc_o \quad . \quad . \quad . \quad . \quad (12)$$

or

$$c_o = ks_o \quad . \quad . \quad . \quad . \quad (13)$$

where

$$k = \frac{A}{t} \quad . \quad . \quad . \quad . \quad . \quad (14)$$

The allowable total load for such a tube is

$$P = pr = c(t + mA) + m_o c_o A \quad . \quad . \quad (15)$$

And the ultimate strength

$$1{\cdot}5P = p'r = c_u(t + mA) + m_o c_o A \quad . \quad . \quad (16)$$

As will be shown later the value of m_o must be taken as at least 60.

From the equations (15) and (16) it is possible to obtain the amount of initial compression in the concrete which is necessary to give the allowable load a factor of safety of 1·5. By expressing that

$$\begin{aligned}1{\cdot}5(c(t + mA) + m_o c_o A)\\ = c_u(t + mA) + m_o c_o A\end{aligned}$$

and introducing k from (14) it is found, by simplying the equation, that

$$c_o = (2c_u - 3_c)\frac{1 + mk}{m_o k} \quad . \quad . \quad (17)$$

And if, in addition, it is required that the allowable tensile stress of the concrete should have a factor of safety of 2·5 as against the ultimate tensile stress, c_o is found from

$$c_o = (2c_u - 1{\cdot}2c_u)\frac{1 + mk}{m_o k}$$

or

$$c_o = 0{\cdot}8c_u\frac{1 + mk}{m_o k} \quad . \quad . \quad (18)$$

The term $\frac{1 + mk}{m_o k}$ becomes, for percentages of reinforcement of 1 and 2 respectively,

$$\frac{1 + 0{\cdot}01 \times 45}{0{\cdot}01 \times 60} \text{ and } \frac{1 + 0{\cdot}02 \times 45}{0{\cdot}02 \times 60}$$

or 2·4 and 1·6.

For the two qualities of concrete, with c_u of the value of 215 lbs. and 350 lbs. per sq. in. respectively the necessary initial compression stresses in the concrete c_o become the following:—

		$c_u = 215$ lbs. per sq. in.	$c_u = 350$ lbs. per sq. in.
(19)	$k = 1\%$	410 ,,	670 ,,
	$k = 2\%$	340 ,,	550 ,,

and the corresponding initial tension stresses s_o in the steel

		$c_u = 215$ lbs. per sq. in.	$c_u = 350$ lbs. per sq. in.
(20)	$k = 1\%$	41,000 ,,	67,000 ,,
	$k = 2\%$	17,000 ,,	27,750 ,,

These equations are, however, to be taken in the sense that the initial stresses need not necessarily be exactly those found above; but if it is required that the concrete stresses in the tube should not reach a greater magnitude than that taken as permissible at the moment when the permissible tube load is reached, the values for the initial tension should be at least those given above. The equation (17) affords a starting point for estimating the percentage of reinforcement, or the initial steel stress, necessary to prevent the occurrence of too high tension stresses in the concrete. If it is desired to increase the strength of the tube by increasing the reinforcement percentage, this will also enable higher initial pressures to be used. The steel reinforcement should always be fully utilised and the initial steel stress should be of such a magnitude that, when increased by the internal pressure in the tube, the total does not exceed the tensile strength of the steel. The ultimate steel stress s_u should therefore be

$$s_u = s_o + m_o c_o + m c_u = s_o(1 + m_o k) + m c_u \quad . \quad . \quad . \quad . \quad . \quad (21)$$

from which is found the initial stress

$$s_o = \frac{s_u - m c_u}{1 + m_o k} \quad . \quad . \quad . \quad (22)$$

or with concrete having a c_u of 215 lbs. and 350 lbs. per sq. in. respectively and a reinforcement of 1 per cent. and 2 per cent.

	$c_u = 215$ lbs. per sq. in.	$c_u = 350$ lbs. per sq. in.	
$k = 1\%$	$\frac{s_u - 9{,}700}{1{\cdot}6}$	$\frac{s_u - 15{,}750}{1{\cdot}6}$	(23)
$k = 2\%$	$\frac{s_u - 9{,}700}{2{\cdot}2}$	$\frac{s_u - 15{,}750}{2{\cdot}2}$	

These calculations will be reverted to in the following, but for the present it may be noted that the equations (12) and (18) supplement each other, and also that it would appear necessary to use high tensile steel in cases where a small percentage is used together with a good quality concrete.

In order to test the correctness of the equation (15) with the theoretical assumption therein contained, the experiments described in the following were carried out. The tests were subdivided into two groups, one dealing with tubes of comparatively small diameter and a spirally-wound wire reinforcement, the other comprising large tubes reinforced with separate rings. The experiments are being continued, and the following description is only a preliminary report in which the results of the tests on the smaller tubes will be treated first.

(*To be concluded.*)

CONCRETE IN AGRICULTURE.

THE Concrete Utilities Bureau proposes to hold two essay competitions, the subject being "The Use of Concrete on Farms and Estates." Competition "A" is limited to students of agricultural and technical schools and colleges in Great Britain and Ireland. (Ex-students who have attended such schools and colleges during any part of 1923 may compete in this class.) Competition "B" is open to all who are not students of the institutions mentioned above. The prizes in each competition will be as follows: 1st prize, £20; 2nd prize, £10; 3rd prize, £5; and to five honourable mentions £1 each. *The Competitions close on December* 31, 1923. "Conditions of Competition" and entrance form may be obtained on application to The Concrete Utilities Bureau, 35 Great St. Helens, London, E.C.3. There is no entrance fee.

The Earthquake in Japan.

HIS Majesty's Chargé d'Affaires at Tokio reports that, according to a statement of the Japanese Minister of Foreign Affairs, it has definitely been decided to reconstruct Yokohama as a port, the damage to the harbour works having proved less extensive than was originally estimated. The Commercial Attaché to the Japanese Embassy in London has informed the Department that repairing work is already in progress at Yokohama and it was hoped that by the end of October ordinary port facilities would be re-established. At Kobé special arrangements have been made to deal with the congestion created by the diversion of vessels to this port, and the situation is improving.

REINFORCED CONCRETE IN RESERVOIR CONSTRUCTION.

By E. R. MATTHEWS, M.Inst.C.E., F.R.S.E.

SUITABILITY OF THE MATERIAL.

THERE is no branch of engineering in which reinforced concrete may be used with better results than in waterworks engineering; in fact it was first used in this manner by that well-known French engineer Monier in the construction of tanks and tubs for storing water for garden purposes.

It has been so extensively used in reservoir construction during the past twenty years that one may say confidently, knowing the material so well, that it is suitable for use in the building of very large reservoirs as well as small service reservoirs. The whole structure may be of reinforced concrete, or the walls only, and the floor of mass concrete; the pillars may be reinforced or of plain concrete, and the partitions and roof of reinforced concrete.

WATERTIGHTNESS OF CONCRETE.

Some engineers have refrained from using the material as they fear that as the walls are so thin they cannot be watertight. There are no grounds whatever for such fears. Many tests have been made, usually with a view to ascertaining whether or not steel embedded in concrete is subject to corrosion from moisture. The writer has made numerous tests of this nature, and it has been found, speaking generally, that rusting of iron only occurs under the combined action of moisture and carbon dioxide.

A coating of Portland cement will not only exclude moisture and carbon dioxide, but in hardening will absorb any carbon dioxide present. If the aggregate is carefully graded and properly mixed under skilled supervision, an absolutely watertight concrete will result.

In America among other methods the following is one often used. "Liquid asphalte is mopped on to the face of the concrete (say the face of the wall); and this forms a coat upon the surface of the unset concrete, the thickness of the asphalte varying from ¼ in. to ½ in. This method is open to the objection that asphalte when exposed to the action of water has a very short life." *

Watertight concrete can (in the writer's opinion) be obtained where a most careful selection is made of the aggregate. This should be carefully graded, and the proportions wisely chosen. The greatest care should be exercised in mixing and placing, and (if for reservoir work), a fairly wet mixture should be used. There is no need to adopt any other method to obtain impermeability.

Some of the materials used in waterproofing in the U.S.A. are: Asphalte (refined), felt (asphalte treated), felt (coal-tar treated), alum, asbestos, benzine, bricks (lining), coal-tar pitch, gasoline, graphite, hydrated lime, lime (common), linseed oil, cement rendering, paraffin oil, soap, turpentine, brick and asphalte mastic, alum and soap.†

It is sometimes advisable to render the inside face of walls of a service reservoir with, say, a 1-in. cement rendering (1 cement to 2 sand), but the writer has found it of greater value to point up the face of the wall with a mixture of one of cement to one of sand, and to cement-wash the surface after the pointing has hardened.

If a good cement and aggregate are employed, the proportions not exceeding 1 : 2 : 4, and the materials are carefully graded as suggested, the walls and floor will be found to be quite watertight.

Wherever watertight work is necessary in the construction of reservoir walls or tanks a "wet" mixture is always preferable to a "dry" one.

"Cinder" concrete in reservoir work should be avoided under all circumstances.

When a 1 cement, 2 sand, and 4 stone mixture has been found to be permeable (which is a rare occurrence) it has been due to one or more of the following defects:—

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* "Reinforced-Concrete Construction." By Adams and Matthews.

† "Waterproofing Engineering." By Joseph Ross.

(*a*) Defective proportioning, grading, and placing.

(*b*) The use of imperfectly-sized and graded aggregates.

(*c*) The use of excessive water.

(*d*) The lack of proper provision for dealing with expansion and contraction, causing cracks.

There is no reason why any of these conditions should prevail. As regards (*d*) it is now definitely established that concrete shrinks whenever it becomes dry, and expands when it becomes wet. These changes in volume are much larger, more rapid, and more frequent than those due to temperature changes.

CONDENSATION CONFUSED WITH LEAKAGE.

If a reservoir is so planned that one side of it is in contact with water (as in the case of a dam) and on the other side there is a free circulation of air, the concrete on the inner face will appear to be absolutely watertight—no water will seep through. If, on the other hand, the air is relatively dead, as in the case of an empty covered reservoir, where no means of ventilation has been made, or in the case of a cellar, it would be found that the humidity of the air in the unventilated reservoir or the cellar would be very high, and moisture would evaporate from the dry surface. Condensation under these conditions would occur.

It has been suggested that if the concrete wall were several feet thick this condensation would not occur.

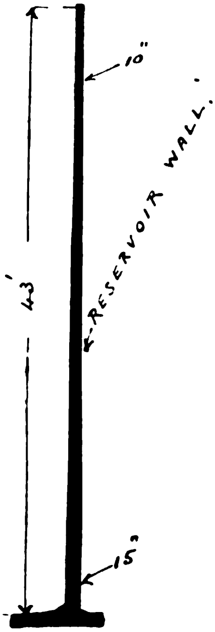


Fig. 1.

A SEVERE TEST.

A good example of a reinforced concrete tank under water pressure is that designed by Mr. W. B. Fuller at Little Falls, N.Y., U.S.A. (*see Fig.* 1). This is a remarkable structure. It is a circular tank, 10 ft. diameter (inside), and 43 ft. high. It was built in record time, and has no surface treatment, and yet it is absolutely watertight. The wall is 15 in. only in thickness at the bottom and 10 in. at the top, and the structure was built in eight hours. It is a perfect monolith. All concrete was poured from the top; and the mixture was very "wet." The proportions were 1 part cement, 3 parts sand, 7 parts broken stone. No plastering of any kind was done, but the tank was found to be quite watertight.

The writer would not recommend in reservoir construction that the proportions should exceed 1 : 2 : 4.

DESIGN OF REINFORCED CONCRETE RESERVOIRS.

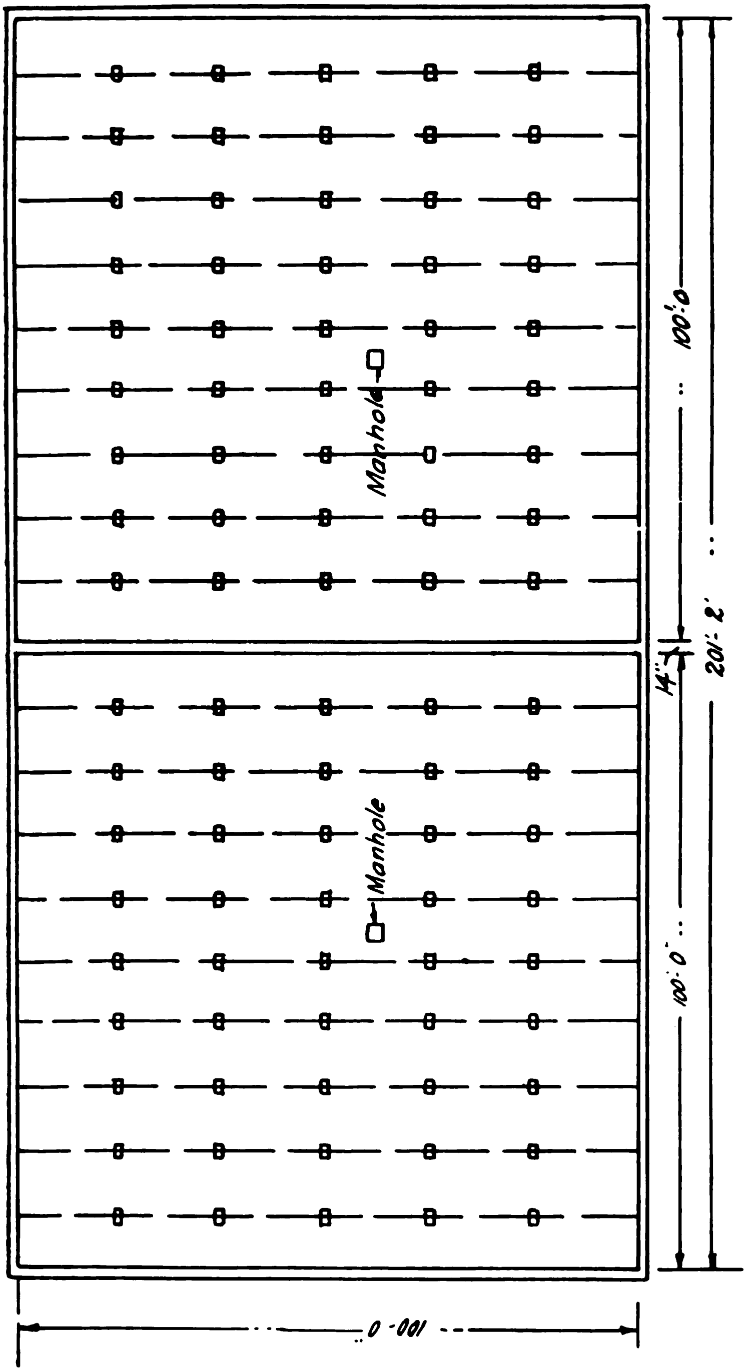
Reinforced concrete reservoirs may be built in plan either circular or rectangular. The first named has the advantage that it has no joints, and it is at joints that leakage usually occurs.

The rectangular reservoir is more often adopted in this country (the circular is the American method). The rectangular structure is usually so designed that filter beds are built on two sides of it, the reservoir itself being sandwiched in between the two sets of filter beds; this is a very convenient arrangement, as it makes the planning of the filter beds quite an easy matter. It is also more easy to arrange the various pipes, valves, overflows, etc.

Reservoir walls may be built with or without buttresses. Where they are necessary they should be constructed outside the walls. Pillars supporting the roof may be circular or square in plan and reinforced as in *Fig.* 2 or of plain concrete. It is the custom in British practice for them to be rectangular with chamfered edges.

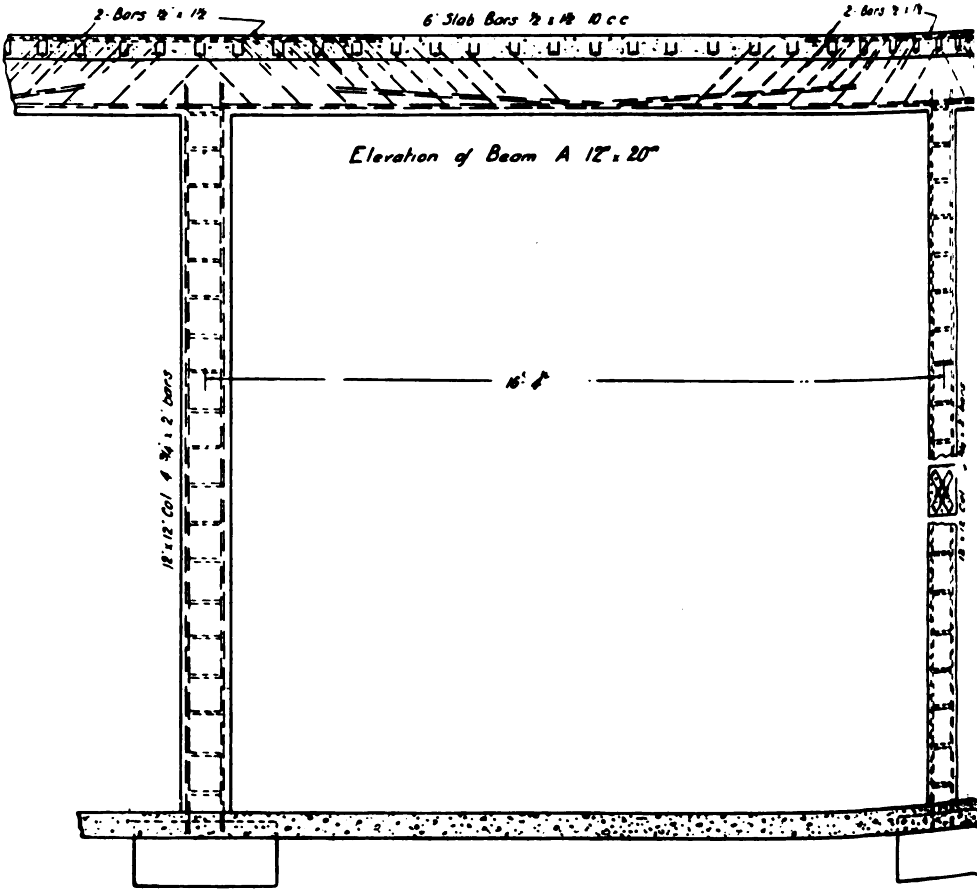
The height of a reinforced service reservoir from floor to under-side of roof will vary, but it is usual to allow for a depth of water of about 15 ft. and for a span between centres of about 16 ft.

Fig. 3 is a plan of a rectangular reservoir where there are no filter beds. *Fig.* 4 shows, in detail, the arrangement of the reinforcement in the roof slab, beams,



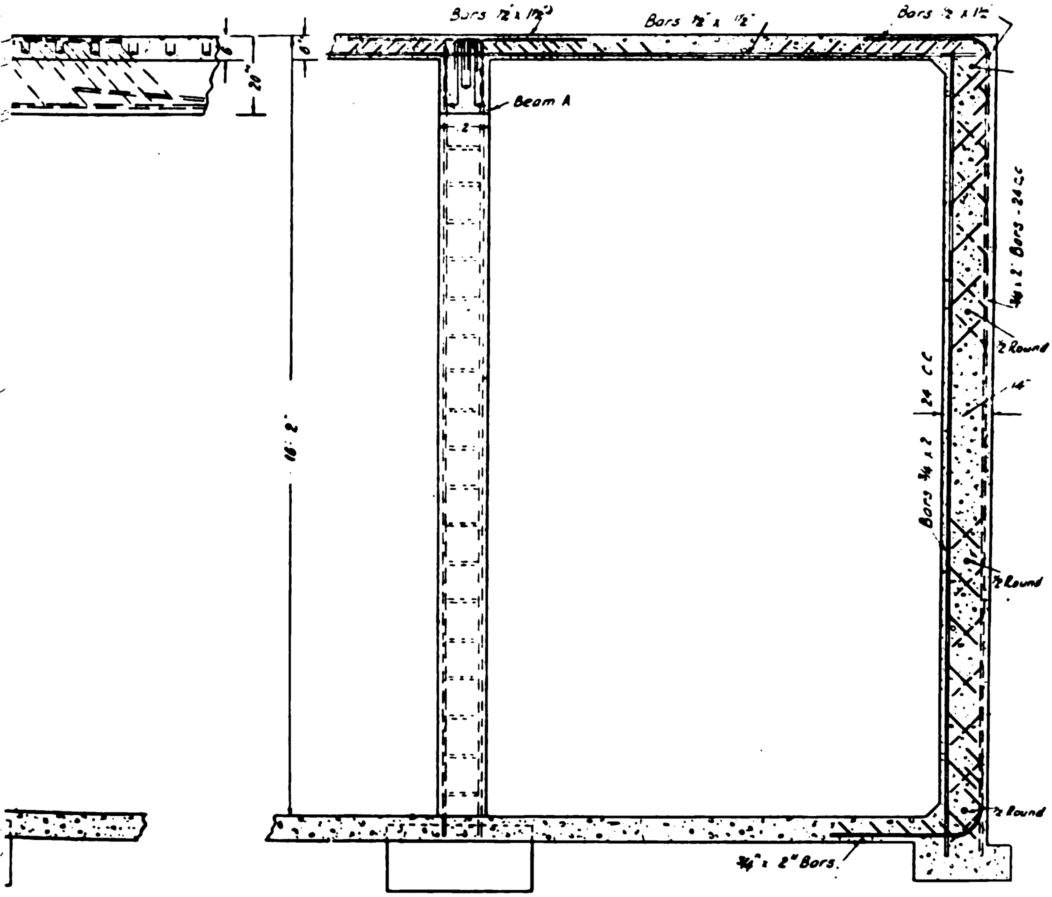
PLAN OF TWO COMPARTMENTS

FIG. 3.



2-Bars ½" x 1½
6" Slab Bars ½ x 1½ 10 c c
2-Bars ½ x 1½
Elevation of Beam A 12" x 20"
16' 4"
12" x 12" Col 4 ¾" x 2' bars

FIG. 2.—REINFORCED CONCRETE



IN RESERVOIR CONSTRUCTION.

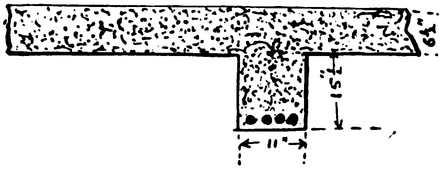


FIG. 4.

pillars and walls in this reservoir, which was constructed in America. The columns look "light" but the 12 in. by 12 in. measurement is taken around the reinforcement; a covering of 1½ in. of concrete must be added on each side to this, making the size of pillar in elevation 15 in. by 15 in. The reinforcement used in America is often square or rectangular, but in this country the round bar is generally used.

PRELIMINARY DESIGN AND APPROXIMATE ESTIMATE.

If the engineer is desirous of submitting a sketch plan showing a client how the work can be carried out in reinforced concrete more economically than in other materials, it is best for him in the first instance to pencil out his scheme, and give his estimate to the client. He would not go into detail in the design, and he might work on the following general formulæ when designing, say, a reinforced concrete rectangular service reservoir (covered).

Loads.[*]

Concrete, 135 lb. per cu. ft. (Often taken at 140.)

Earth, 100 lb. per cu. ft.

Live Load, 120 lb. per cu. ft.

Working formulæ in this case :—

Suitable values are :

$$m = \frac{Et}{Ec} = 15.$$

$c = 600$ lb./in.^{2}

$r =$ proportion of reinforcement 1½ per cent.

These give $n^1 = \cdot483$.

$$\text{and } bd^2 = \frac{M \text{ in. lb.}}{121\cdot48 \text{ lb./in.}^2}$$

Roof Slab.

$$\text{Max. B·M.} = \frac{Wl}{12} \text{ at centre.}$$

Taking 13 ft. span between roof beams and considering 1 in. width over it.

Weight of concrete =

$$\frac{6\frac{1}{2}}{12} \times \frac{1}{12} \times 13 \times 135 = 79\cdot00 \text{ lb.}$$

(Assumed provisional thickness 6½ in.)

Weight of earth covering =

$$\frac{9}{12} \times \frac{1}{12} \times 13 \times 100 = 81\cdot25 \text{ lb.}$$

Extra load

$$\frac{1}{12} \times 13 \times 120 \qquad = 130\cdot00 \text{ lb.}$$

$$W = 290\cdot25 \text{ lb.}$$

$$\therefore M = \frac{290\cdot25 \times 13 \times 12}{12} = 3{,}775 \text{ in. lb.}$$

$$\therefore d^2 = \frac{M}{121\cdot48b} = \frac{3{,}775}{121\cdot48 \div 1} = 31\cdot0^2 \text{ in.}$$

$\therefore d = 5\cdot56$ in., say 6½ in. Take 1 per cent. reinforcement. Area steel

$$= \cdot015 \times 5\cdot56 \times 1$$
$$= \cdot0835 \text{ per in.}$$

Use ¾ in. rods at 5 in. centres.

Roof Beams.

Span 14 ft. between columns.

Greatest area per span =

14 ft. × 13 ft. = 182 sq. ft.

Weight of concrete

$$\frac{6\frac{1}{2}}{12} \times 182 \times 135 \qquad = 13{,}300 \text{ lb.}$$

Weight of earth

$$\frac{9}{12} \times 182 \times 100 \qquad = 13{,}660 \text{ lb.}$$

Extra load

$$182 \times 120 \qquad = 21{,}800 \text{ lb.}$$

$$W = 48{,}760 \text{ lb.}$$

$$\therefore bd^2 = \frac{48{,}799 \times 14 \times 12}{12 \times 121\cdot48} = 5{,}623 \text{ in.}^3$$

Take $b = 12$ in.

Then $d = 21\cdot6$ in., say $\underline{\underline{22 \text{ in.}}}$

and depth of beam below slab

$= 22$ in. — 6½ in. $= \underline{\underline{15\frac{1}{2} \text{ in.}}}$

Reinforcement, say 4 bars 1⅛ in. in diameter.

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[*] See graph 10, page 28, Dr. Faber's book, Vol. I, and graph 16, page 37.

This gives

$$\textit{Reinforcement} = \frac{4 \times \frac{\pi}{4}(1\tfrac{1}{8}\text{in.}^2)}{22 \times 12} = 1{\cdot}51\%$$

Note.—It is usual to insert less reinforcement, 1 per cent. being a useful quantity.

The breadth of beam is generally taken as half the depth.

Outside Wall of Reservoir.

This may be taken as a cantilever (if we neglect the stiffening effect of the roof) fixed at the base, so that the B.M. about the base is :—

$$\frac{wbh^3}{6}$$

viz. :—

$$\frac{62\tfrac{1}{2} \times b \times (12\tfrac{1}{2})^3}{6} = 121{\cdot}48bd^2.$$

$$\text{Therefore } d^2 = \frac{20{,}345}{121{\cdot}48} = 167{\cdot}48.$$

Whence $d = 12{\cdot}92$ in., say 13 in.

With 1½ in. cover for reinforcement

$$d = 14\tfrac{1}{2} \text{ in., say } 15 \text{ in.}$$

Width at top 8 in.

Floor.

This may be of reinforced concrete or plain concrete (preferably the former) and the manner of calculating it is very simple.

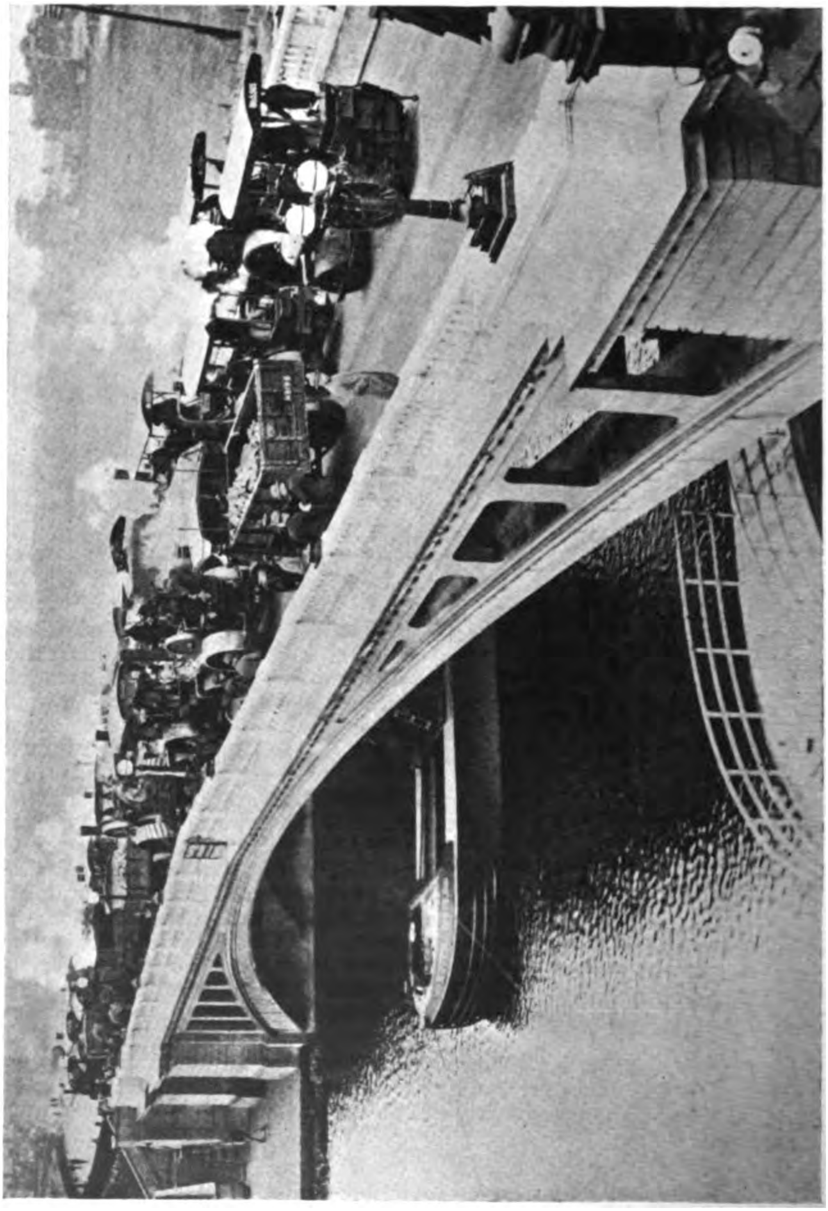
Generally :—The cost of excavation, filling, and many other items will be common to all schemes.

No brick lining is necessary in the reinforced concrete scheme.

THE LATE MR. H. T. DESCH.

Mr. H. T. Desch, who died recently, was the only son of the late Henry Charles Desch, and was born in London in 1849. He was educated in Northamptonshire, and returning to London at the age of 16 was articled to the late Mr. Cubitt Nicholls, as a quantity surveyor. After completing his articles he took a post as surveyor in the firm of Messrs. Wm. Cubitt & Co., remaining with that firm up to the time of his death. He took a great interest in reinforced concrete work when first introduced into this country, and on behalf of his firm carried out several important contracts in this form of construction in London and the provinces. On behalf of his firm he was associated with the carrying out of many important contracts in London, particularly around the Bank of England, such as the offices of the Commercial Union Assurance Co., Lloyds Bank, New York Mutual Life Insurance Co., Scottish Widows Fund, Scottish Provident, Bank Buildings, Grocers' Hall, etc.; also Messrs. Verners Flour Mills at Victoria Docks, the Cunard Building at Liverpool, and Mount Nelson Hotel, at Cape Town, and other contracts, both at home and abroad. Some four years ago, in recognition of his services over such a long period, the directors of the combined firms of Messrs. Holland & Hannen & Cubitts, Ltd., appointed him a member of the Board. At the time of his death Mr. Desch had been associated with the firm for some 52 years.

Mr. Desch is survived by his widow, two sons and a daughter. The elder son, Professor C. H. Desch, F.R.S., Professor of Metallurgy at Sheffield University, has been engaged for some years past on research in connection with the preservation of stone from decay.



REINFORCED CONCRETE BRIDGE AT READING: SHOWING TEST LOAD. (*See* p. 717.)

REINFORCED CONCRETE BRIDGE AT READING.

THE new bridge illustrated in this issue, which spans the Thames at Reading, was opened to traffic last month. The bridge is constructed on the Hennebique system of reinforced concrete, the designers and engineers for the work being Messrs. L. G. Mouchel and Partners, of Westminster. The contractors were Messrs. Holloway Brothers (London), Ltd.

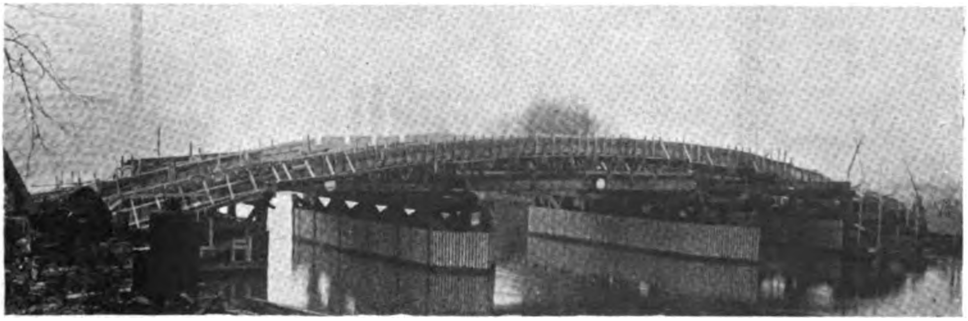
On the southern bank the slope is 1 in 30, and on the northern 1 in 35, the east side terminating at Macduff's Hotel, and that on the west swinging round at right angles, nearly opposite the hotel, and being continued for some distance along the street past some stores. Access to the tow-path on the south side of the river is provided by steps, while on the north bank the bridge is terminated by splayed wing walls of comparatively short length, the approach being completed by a raised earth embankment with sloping sides not confined by walls. At the north abutment a stairway to normal ground level is also provided.

The main span is 180 ft., and the rise 18 ft. The southern approach rises on a series of arches of 19 ft. 6 in. span carried on piers standing on wide concrete bases; the lower portion of the approach is plain filling confined between walls finished with false arches and piers, the latter being centred 16 ft. apart. This spacing is continued for the piers in the spur wall. Immediately in the rear of each abutment is a single semicircular archway of 12 ft. width.

The width of the bridge between parapets is 40 ft., with a roadway of 27 ft. and two footpaths of 6 ft. 6 in. As regards exterior finish, reliance for effect has been placed in the adoption of bold mouldings for the cornices, parapets, etc., and the treatment of the main piers. The parapets and abutments are of Portland stone, the other parts of the structure, including exterior work, being of reinforced concrete.

The abutment foundations are mass concrete blocks about 50 ft. long by 45 ft. wide. The bulk of the concrete in these blocks is of 1·11 : 3 : 6, but at the inside upper edge taking the direct thrust of the arch 1·5 : 2 : 4 concrete is used for a depth of about 9 ft. by 7 ft. across the block, strongly reinforced. Longitudinal reinforcing bars of 1 in. diameter at 12 in. centres are also embedded in the mass concrete blocks near the lower surface. The concrete for the arch ribs was in the proportions of 1·5 : 2 : 4, and the remainder of the concrete 1·11 : 2 : 4.

The main span is formed of four main arch ribs supporting longitudinals which carry secondary beams and the decking. Four sets of horizontal braces are arranged between the main arch ribs. The arch ribs run out into the main longitudinals and decking as the middle of the arch is approached, but the portions of the span where they are widely separated are furnished with spandrel columns connecting the two. The main outside ribs are 48 in. wide and 56 in. deep at the springings. The depth reduces to 31 in. at the crown. The two inner ribs are 54 in. wide and 42 in. deep at the crown. At the springings the depth is the same as for the outer ribs. The ribs

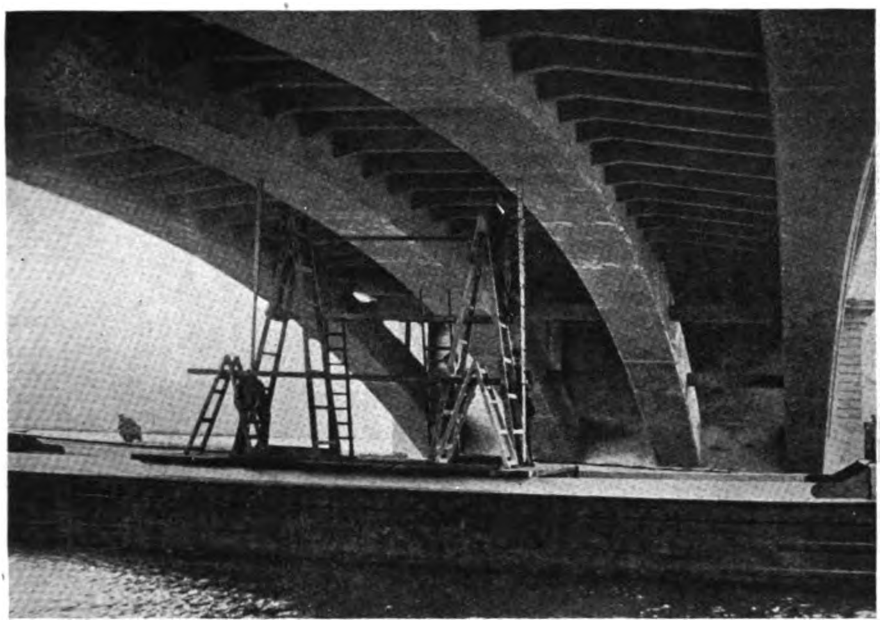


REINFORCED CONCRETE BRIDGE AT READING: DURING CONSTRUCTION.

are centred 12 ft. 9 in. apart. The spandrel columns are 10 in. by 24 in., except at the abutment, where the inside columns project 16 in. only from the vertical face of the 4½-in. cross-wall. The columns are centred 10 ft. apart longitudinally, the interior rows being placed centrally on the arch ribs, while those carried by the outer ribs are placed at the outer face of the latter. The spandrel columns support the main longitudinals, which are 10 in. wide by 20 in. deep apart from the deck slab, which is 7½ in. thick. The secondary beams are spaced at 5 ft. centres; they are 7 in. wide and of 14 in. depth below the decking. Where the longitudinals run out into the arch ribs the secondary beams extend direct between the latter, and form a stiff structure. In the spandrel lengths of the arch where the ribs do not benefit by the support, in a sideways direction, of the decking, braces have been introduced of a section of 14 in. by 14 in. The reinforcing of the main ribs consists of 1¾ in. bars, the inside ribs having 14 bars both at the top and bottom, this number being increased to 18 at the crown. In the case of the outer ribs 7 bars are used, top and bottom, near the haunches, increased to 18 top and bottom at the crown. The underside of all the arch ribs is lined off uniformly, the greater depth of the central ribs at the crown being accommodated by raising the central portion of the decking to the extent of 11 in. The main bars of the reinforcement in the arch ribs are anchored in the mass concrete foundation. The outer ribs and longitudinals incorporate cornice mouldings and carry the stone parapets with open balustrading.

On the southern side, for a distance of 142 ft., the road is formed over ballast and chalk filling placed between reinforced concrete retaining walls carried on mass concrete footings, from which reinforced uprights and counterforts are carried up at intervals, supporting thin reinforced concrete wall panels. Beyond this the approach takes the form of a raised viaduct, which consists of a series of spans on piers centred 19 ft. 6 in. apart. It is slightly curved, but the main components are of uniform design. The spans are carried on piers placed three in a row across the bridge. The main beams connecting the columns or piers above the base have a section of 1 ft. 6 in. by 3 ft. 6 in., in a longitudinal direction. These columns are centred 20 ft. 7½ in. apart across the bridge. The main beams are 12 in. wide by 3 ft. 6 in. deep and the main reinforcement consists of three 1½ in. bars and three 1⅜ in. bars, additional bars being inserted in the lower part of the beam under the pipe channels. At the piers the beams are increased in depth by some 9 in.

The longitudinals consist of two exterior arched beams and of seven straight beams. The main arc is struck at a radius of 14 ft., running out into the piers through arcs struck at 2 ft. radius. The reinforcement consists of three main bars above and four below, the cornice having a further two bars embedded in it. In addition to the usual stirrups extending between upper and lower bars, bent transverse ¼-in. reinforcing bars are inserted at 8-in. intervals in these beams. The seven inside longitudinals of each bay are of three types, the three in the centre being uniform, and the two outermost being of heavier section and reinforcement. These beams are 12 in. wide, compared with 8 in. for those further inside, and the reinforcement consists of six bottom bars and three top bars, as against four bottom bars only for the other beams. The whole of this end of the approach is based upon the large mass concrete foundations which take the thrust of the main span. These consist of monoliths measuring 50 ft. 6 in. in the longitudinal direction, and extending to a depth of about 26 ft. below ground level. At this end of the viaduct the bays between the piers are filled in with side walls, and the viaduct ends in a cross wall, 9 in. thick in its lower portion and 7 in. thick above, stiffened by four verticals 1 ft. by 2 ft. 1 in. and 2 ft. 3 in., reinforced with 1⅛-in. and 1¼-in. bars, and an intermediate horizontal of 10 in. by 1 ft. 11 in. section, reinforced by 1⅛ in. bars. The base of this cross-wall is a reinforced toe, 2 ft. 3 in. wide and 12 in. thick, formed integral with the mass concrete foundation. This wall takes the thrust of the semi-circular archway, which has a thickness of 7 in. and is reinforced with ⅜-in. bars. On the opposite side this arch is supported by low buttresses. The buttresses are 8 in. wide, and the reinforcing bars are

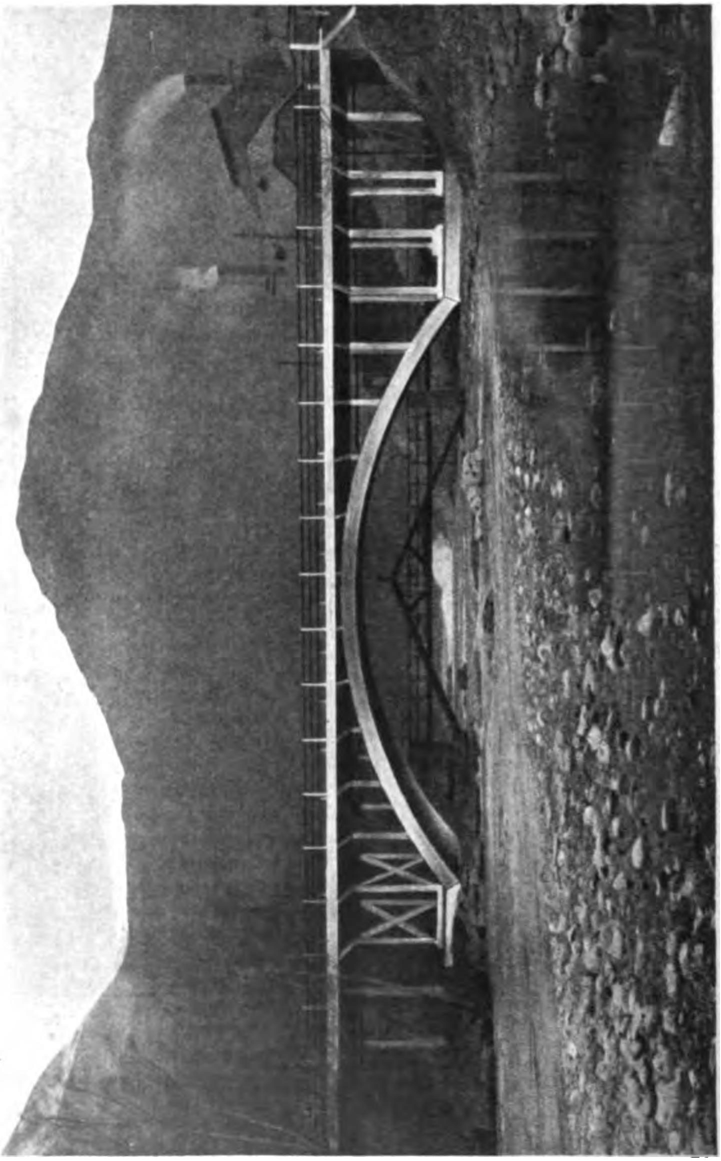


Reinforced Concrete Bridge at Reading.

1 in. and 1⅛ in. in diameter, and run down into the mass concrete of the foundation. In line with the top of the buttresses is a horizontal beam 10 in. deep by 1 ft. 11 in., corresponding to the stiffener on the other side of the arch. The side panels on each side of the 12-ft. archway are 9 in. thick for the lower portion and 6 in. thick above, reinforced with vertical and horizontal ⅜-in. bars inside and out.

On the north side of the river the main span terminates in a similar manner to that adopted on the southern side, and the main mass concrete foundation carries a 12-ft. archway flanked by piers and panel walls. Beyond this the roadway runs out between curved wing walls in the form of a gradually falling embankment. The wings are carried on mass concrete which is wide close to the bridge and gradually narrows down as the height decreases. On the west side incorporated in the wing wall structure is a stairway leading to ground level. This obviates the need for counterforts or buttresses at this point, but the further extension of the wall is provided with two small counterforts. The eastern wing is supported by five counterforts of a height and bottom width varying with the position in the wall.

In the test (illustrated on p. 716) 31 loaded vehicles, including tractors, lorries, steam-rollers, etc., were passed simultaneously on to the bridge. The total weight of the vehicles was 372 tons, and under this loading the deflection at the centre of the arch was less than ¼ inch.



REINFORCED CONCRETE BRIDGE AT KINLOCHLEVEN, ARGYLLSHIRE.

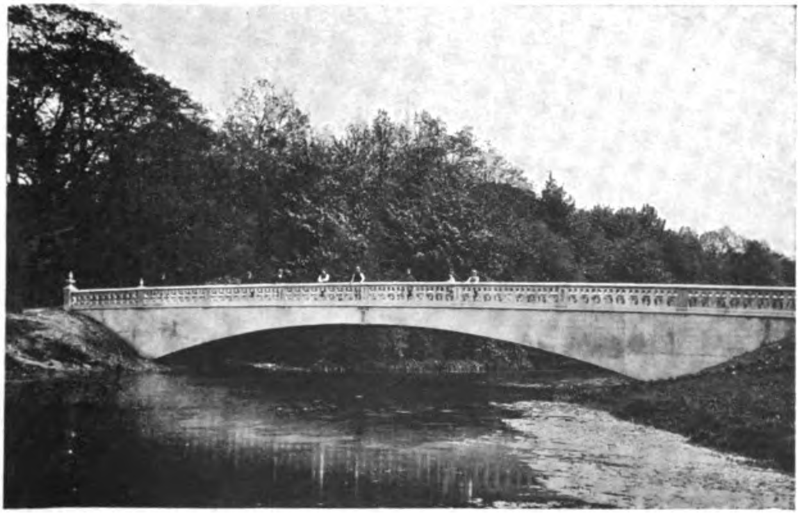
TOTAL LENGTH 180 FT.; SPAN OF ARCH, 100 FT.

REINFORCED CONCRETE ROAD BRIDGES.

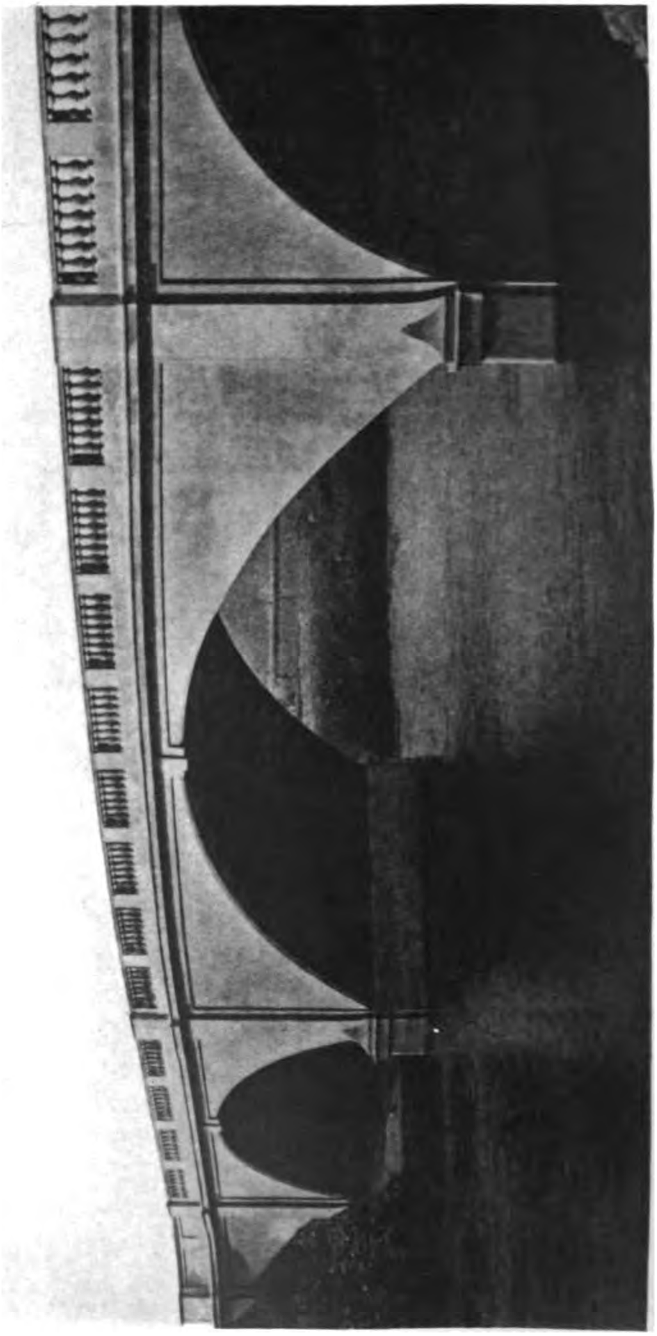
In view of the considerable expenditure which the Government has recently voted for the construction of new roads, it is interesting to consider the question of the new road bridges which will be required in connection with the various schemes which are now being promoted. In many cases it is already anticipated that reinforced concrete will be largely used for the construction of highway bridges, either for entirely new structures or to replace old bridges which have become unsafe or which are no longer strong enough to carry the increasing weight of traffic.

In the past bridges were designed and constructed in accordance with the heaviest rolling loads which were likely to be found in each particular locality, but the recent development of road transport by motor traction throughout the whole country has rendered it necessary that a uniform system of rolling loads should be adopted for all the new bridges. It is for this reason that the Ministry of Transport has now adopted a standard load for highway bridges, which consists of a traction-engine weighing 20 tons, drawing a certain number of trailers, having each a weight of 13 tons. Precise instructions are given by the Ministry as to how this load is to be applied, also the percentage of increase required for impact and all the necessary dimensions between the wheels and axles.

In its "Report on the Road Fund for 1921–22," issued this year, the Ministry of Transport states: "It is obviously desirable for a uniform standard of strength to be applied to bridges on all the principal highways, and with this end in view a standardised loading for highway bridges has been prepared (see diagram). The aim has been to ensure that any bridge, towards the cost of which a grant is made or a loan recommended, shall be so designed as to carry any traffic which may come on the road, so that no restriction need be imposed on the use of the bridge. In deciding upon this standard loading, it had to be borne in mind that a bridge is the most important link in the highway chain, and that a life of at least one hundred years should be anticipated. It was therefore essential that a margin should be allowed to cover the more onerous loading which may be expected in coming years. This standard loading is only applicable to bridges required to carry normal traffic, and in certain industrial districts, such as Liverpool, where heavy indivisible loads in excess of 16 tons have to be dealt with, the bridges must be designed of greater

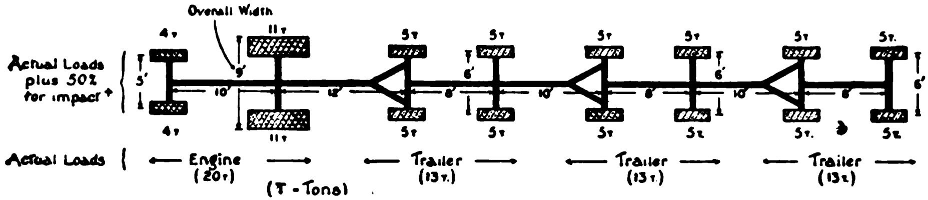


Reinforced Concrete Bridge in Crewe Park.
(Span of Arch 90 ft., width 16 ft.)



CRESSAGE BRIDGE: A REINFORCED CONCRETE BRIDGE IN SHROPSHIRE.

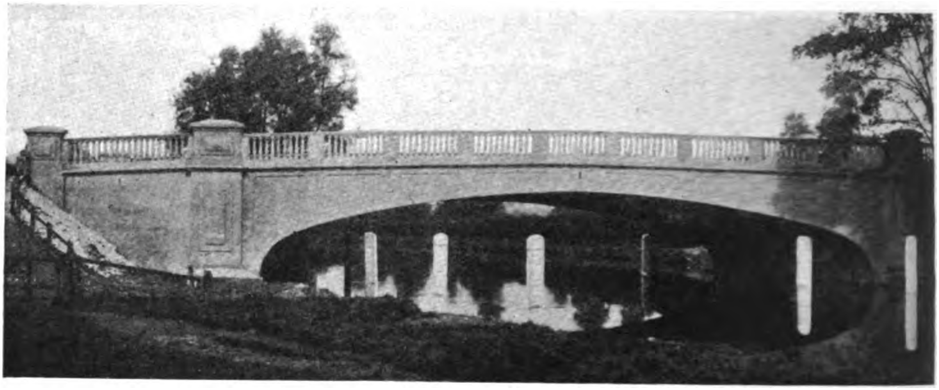
(Centre Span 80 ft.; End Spans 40 ft.)



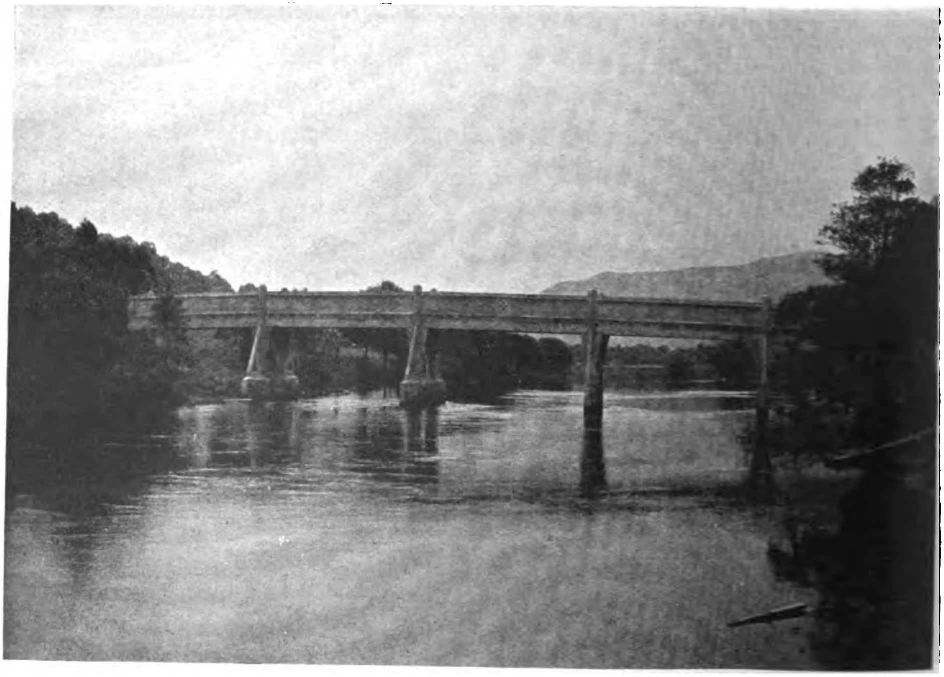
STANDARD LOAD FOR HIGHWAY BRIDGES.

strength accordingly. These exceptional cases fall to be considered, in the first instance, by the highway authorities concerned. The load specified, namely, one traction engine and three trailers, is the heaviest dead load that is permitted for a train of vehicles having steel tyres, and to this dead load 50 per cent. has been added for impact. It must be admitted that 50 per cent. can only be regarded as an empirical figure, for which it is hoped to substitute an appropriate formula as the result of experiments which are now proceeding. The principal difficulty encountered by highway authorities in the matter of bridges arises in the case of those structures which are maintainable by undertakings such as canal companies, railway companies, etc., whose liability is limited to the maintenance of a structure adapted to the loads and traffic conditions of the period in which the bridge was erected. In these days it is evident that the greater number of canal bridges and many railway bridges need entire reconstruction. Not only is their strength inadequate, but in many cases the width is insufficient and the camber of the bridge unsuitable for fast-moving traffic. Continuous pressure is being exercised by the Department, and many conferences have been held with a view to drawing up schemes for the apportionment of the cost of reconstruction between the undertaking, the highway authority, and the Road Fund. In view of the magnitude of the problem, some years must necessarily elapse before the highways of the country, especially in regions traversed by canals, can in this respect be brought up to a standard likely to give complete satisfaction to road users. In connection with the standardisation of bridge design, with regard to carriageways generally, a standard width is now recommended, based on a unit of 10 ft. per line of traffic. This dimension applies equally to the carriageway of highways and of bridges, and its adoption has been generally welcomed by local authorities. The introduction of such a standard will serve a

TEME BRIDGE, TENBURY.
(Total Length 210 ft.; Span of Arches 25 ft. to 38 ft.)



REINFORCED CONCRETE BRIDGE AT WOODBRIDGE, GUILDFORD.
(Span 70 ft.; Width 40 ft.)



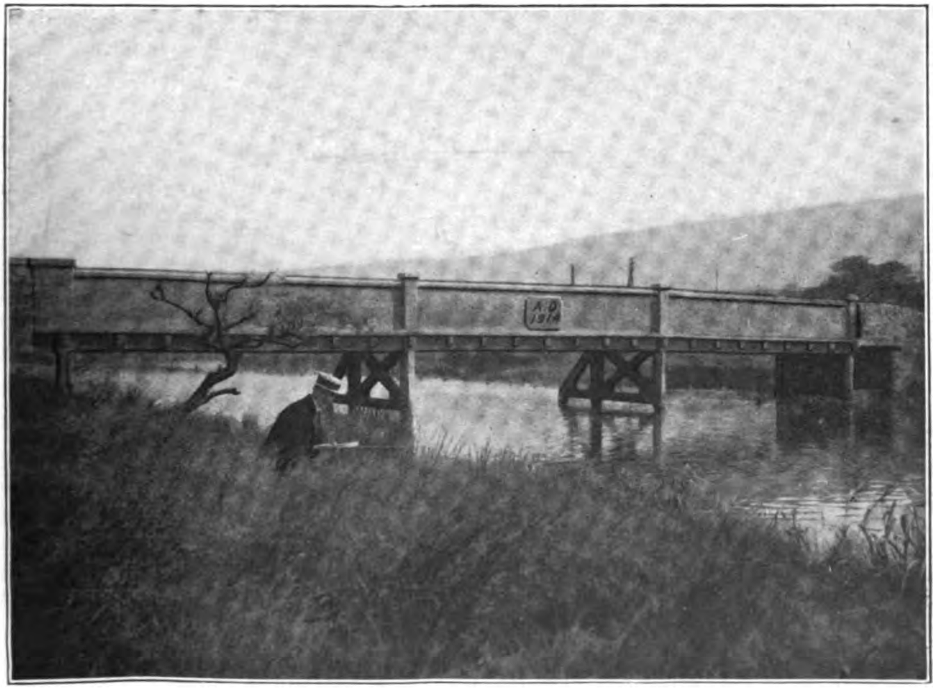
ROAD BRIDGE AT MAULD, NEAR INVERNESS.

useful purpose in discouraging the creation of carriageways of unnecessary width, as it is clearly undesirable that intermediate dimensions should be adopted which give an excessive width for (say) two streams of traffic, and an inadequate width for three."

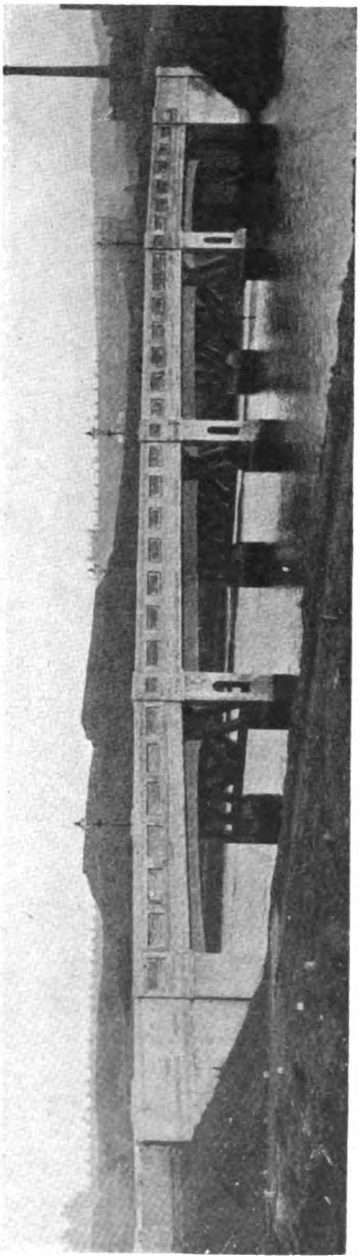
In the diagram, the bridge is assumed to be carrying such standard loads or parts of standard loads as will produce the maximum stress in any bridge member or material under consideration, provided that in any train of loads there shall not be more than one engine per 70 ft. of the span of the bridge, and that the distance between the centre lines of two adjacent trains of loads shall be taken as 10 ft. in normal cases where the width of the carriageway is a multiple of 10 ft. Where a smaller multiple is adopted (but not less in any case than 8 ft.) it shall be assumed that the distance between the centre lines of two adjacent trains of loads is reduced to the dimension of the lesser multiple so adopted. The overall width of 9 ft. shown on the diagram must then be assumed to be reduced so as to leave 1 ft. clearance between the wheels of two adjacent trains.

It will be seen by this that all new bridges must now be designed to support the same rolling load. The only thing the designer has to choose is the particular type of bridge which he will adopt.

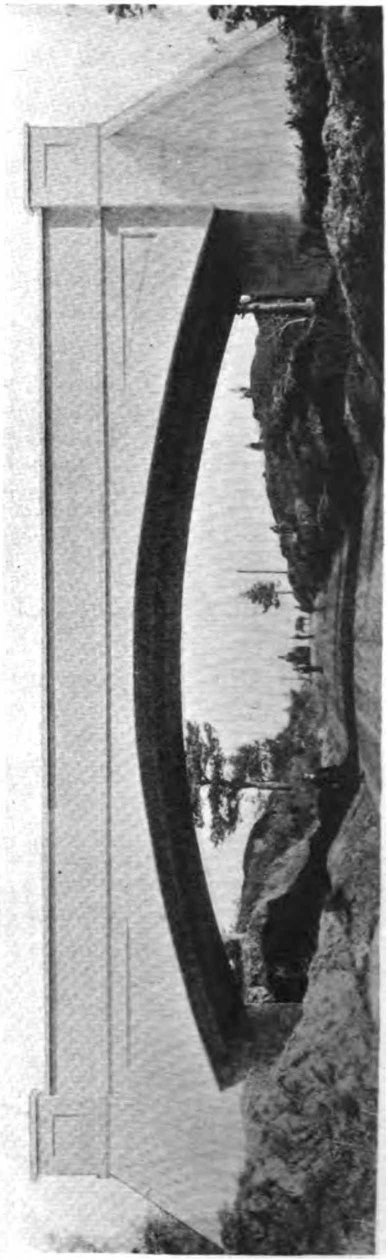
The illustrations in this issue show various types of road bridges suitable for different circumstances. For instance, the reinforced concrete bridge at Sterne Mills, near Halifax, has a total length of about 90 ft., composed of three spans of 30 ft. each. In order to increase the width of the waterway, a small additional span of 10 ft. was left at one side of the bridge to accommodate flood water. In this particular case, owing to the bad nature of the ground forming the bottom of the river, it was necessary to drive reinforced concrete piles to a fairly considerable depth, as it would have been much more expensive to construct coffer dams and carry the foundations down to the solid ground in the bed of the river. For the same reason it was found preferable also to support the ends of the bridge by means of piles instead of ordinary reinforced concrete abutments.



Reinforced Concrete Bridge at Sterne Mills, near Halifax.



REINFORCED CONCRETE BRIDGE AT STRANMILLIS, BELFAST.



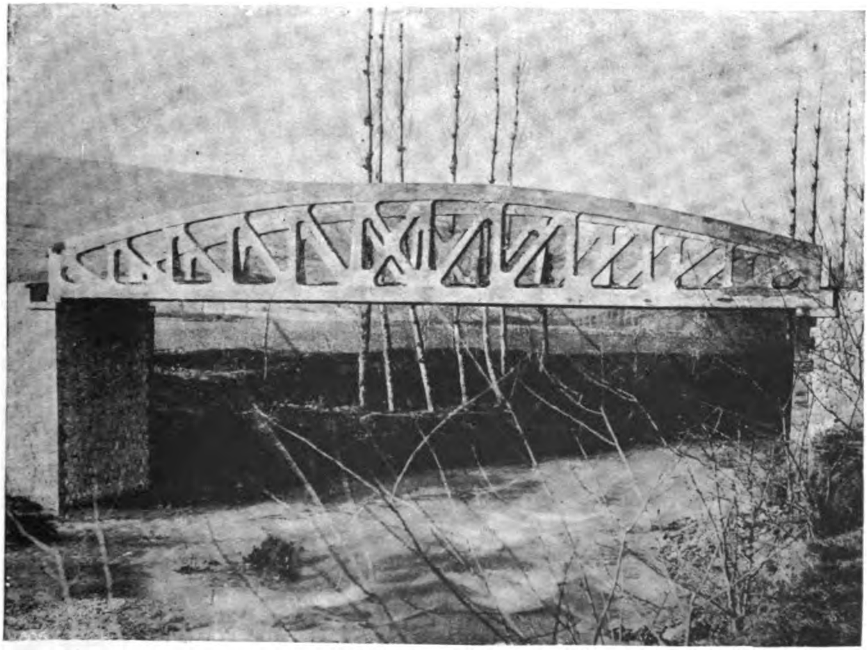
REINFORCED CONCRETE BRIDGE, UPPER PARKSTONE, DORSET.

In this bridge the parapets are used to form reinforced concrete beams, on account of the height of lever arm which can thus be obtained. This arrangement is more economical than if the principal beams had been stopped at the level of the roadway. This method of procedure, however, can only be adopted when such bridges are not too wide and also when there is a sufficient height of waterway to allow the passage of the swollen river in flood-time. The bridge was calculated for heavy road traffic.

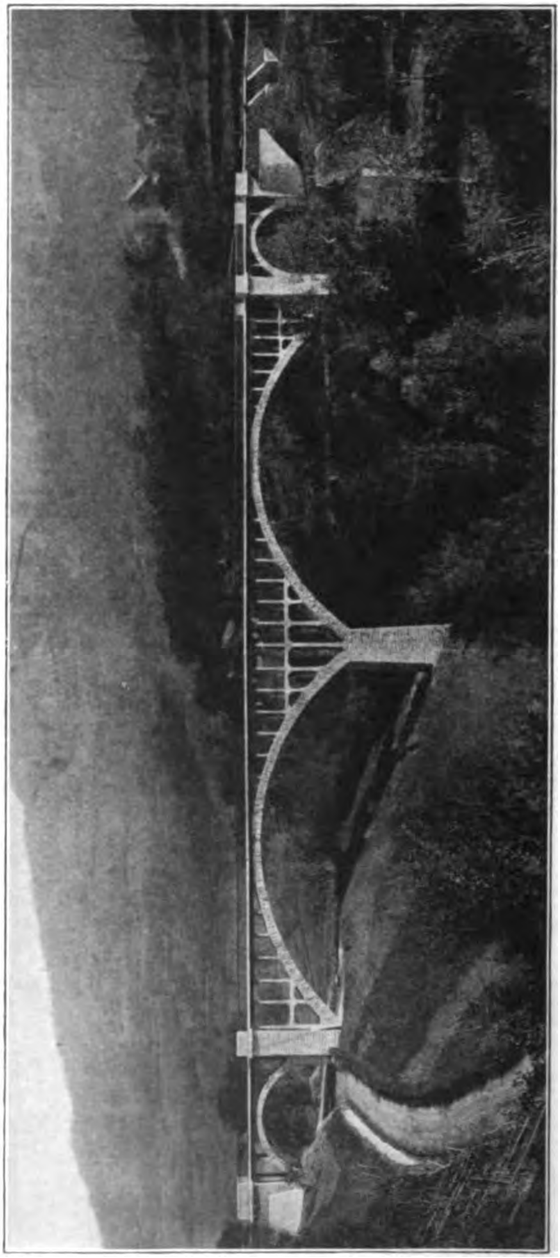
The road bridge at Mauld, near Inverness, has a total length of 180 ft., with a width of 12 ft. between parapets, and was designed for ordinary road traffic. Here again a clear waterway is provided to give a minimum of obstruction during floods.

The question of the passage of storm water and floods is one which affects very seriously the design of country road bridges, and it is for this reason that we herewith illustrate below a reinforced concrete bow-string bridge of a kind which has not been, as far as we know, adopted in this country for highway bridges. It has the advantage of offering the greatest amount of waterway owing to the absence of intermediate piers. The bridge illustrated was constructed in France many years ago and measures 66 ft. in span; it also carries an electric tramway. This type of bridge has been adopted to a certain extent for roads in Italy, where the rivers are often inclined to overflow, carrying away tree trunks and heavy boulders.

In the reinforced concrete bridge constructed over a deep ravine (illustrated on p. 728), the arches are built in mass concrete, and the piers are in masonry. The pillars and beams supporting the roadway and the roadway itself are in reinforced concrete. This bridge was constructed in France by the late Mr. Edmond Coignet, and is a type which is seldom adopted in this country. In districts where the aggregate for the concrete is to be found on the spot, this form of bridge might be found more economical than steel construction, especially if the fact is taken into account that reinforced concrete requires practically no maintenance whatever, whereas a very considerable amount of money has to be spent at comparatively frequent intervals



Reinforced Concrete Bowstring Bridge in France.



REINFORCED CONCRETE BRIDGE OVER A DEEP RAVINE.

in repainting and generally maintaining steel bridges, which in spite of this are liable to deteriorate in time to a very much greater degree than reinforced concrete.

The road bridge at Mauld, near Inverness; the bridge at Sterne Mills, near Halifax, and the Bowstring bridge in France, were designed by Messrs. Edmond Coignet, Ltd., of 125 Gower-street, W.C.

Stranmillis Bridge, Belfast; Ringwood Road Bridge, Upper Parkstone, Dorset; Prince's Bridge, Leamington Spa; and the bridge at Woodbridge, Guildford, were designed by Messrs. The Trussed Concrete Steel Co., of 22 Cranley Gardens, S.W.7.

The following bridges were designed by Messrs. Mouchel & Partners, Ltd., of 36–38 Victoria Street, S.W.1, on the "Hennebique" system: Crewe Park; Teme; Roydon; Kinlochleven; Cressage; Gorgie Road, Edinburgh.

The Indented Bar and Concrete Engineering Co., Ltd., of Queen Anne's Chambers, S.W.1, were responsible for the design of the following bridges:

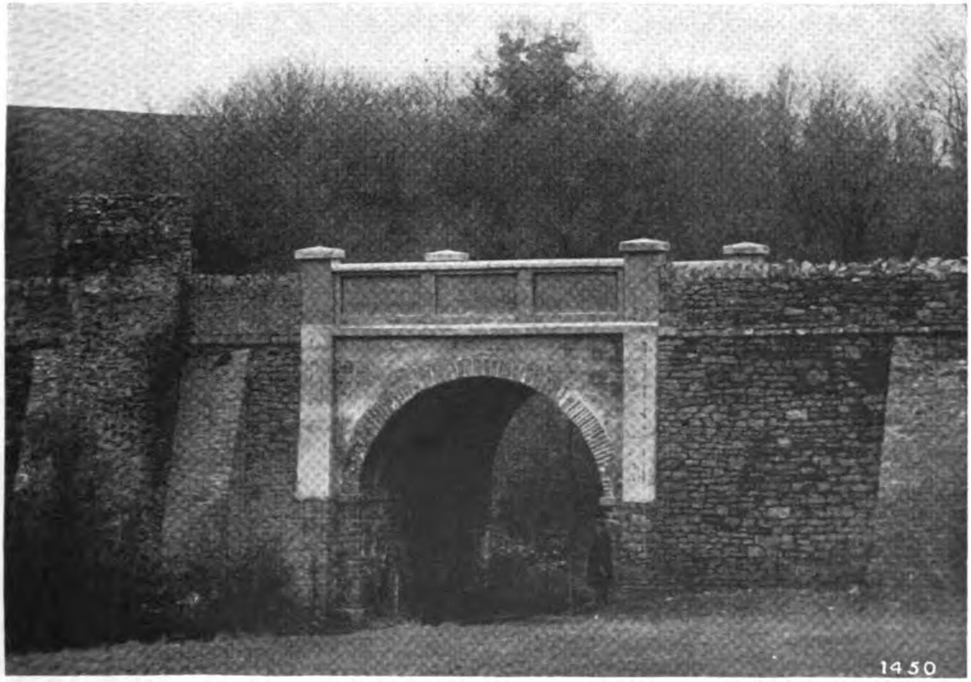
Abercynon: Clear span, 130 ft.; overall length, 164 ft. 6 in.; mean width of roadway, 10 ft.; radius of arched ribs, 156 ft. 8 in.

Doublebois Bridge, Cornwall: Overall length, 70 ft.; clear span of bridge, 34 ft.; width of roadway, 20 ft.

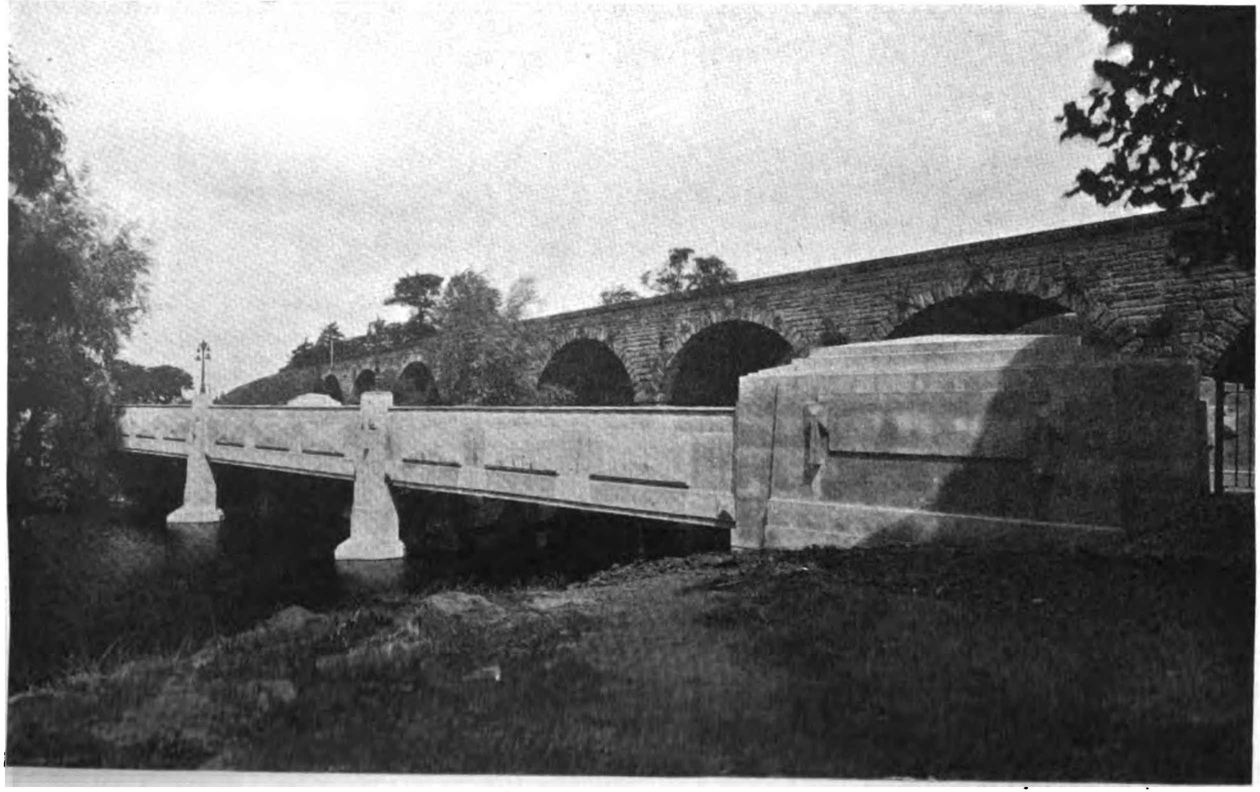
Whittlesey Bridge: Clear span of bridge, 38 ft. 6 in.: width of roadway, 14 ft.

Haberfield Bridge, Easton-in-Gordano: Span, 17 ft. 9 in.; width of roadway, 19 ft. 3 in. This work is particularly interesting as the old arched bridge having been found not strong enough to carry present-day traffic, the reinforced concrete strengthening was inserted on the old abutments over the original masonry arch. The concrete work is independent of the old arch, however, and no stresses are imposed on the existing work. The cost was £981, exclusive of the cost of forming the road.

The following bridges have been recently constructed, or are now in course of construction, to the designs of Messrs, Considère Constructions, Ltd., of 72 Victoria Street, S.W.1—East Ham Bridge: Span, 73 ft. 6 in.; width 26 ft.

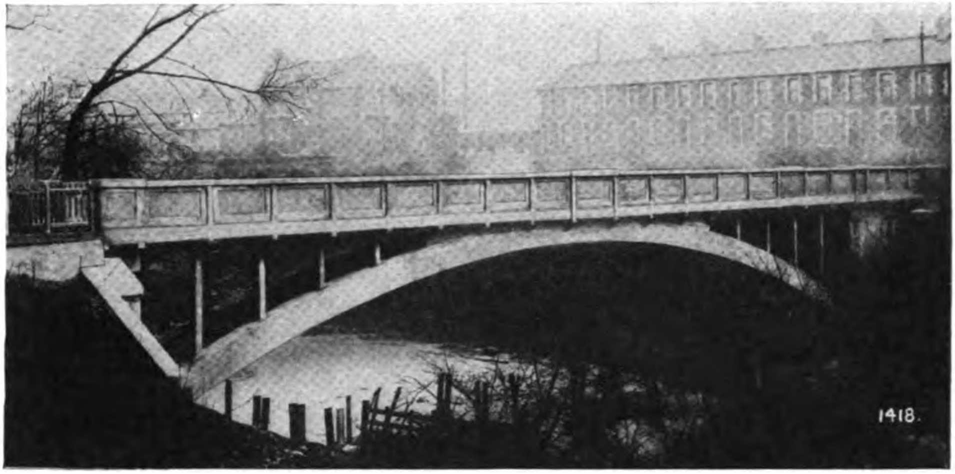


Reinforced Concrete Bridge at Haberfield, Easton-in-Gordano.



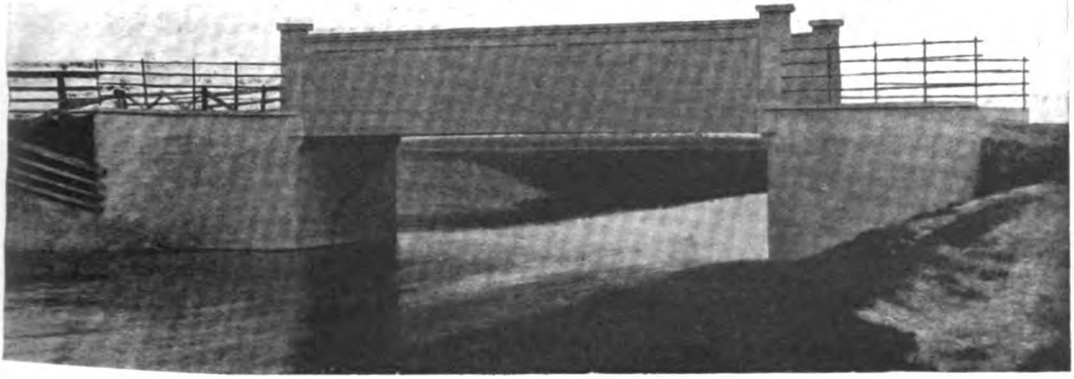
REINFORCED CONCRETE BRIDGE AT LEAMINGTON SPA.

This bridge was tested with two trains of loads passing at the centre of the centre span as specified by the Ministry of Transport, and the deflection was $\frac{1}{740}$ of the span.

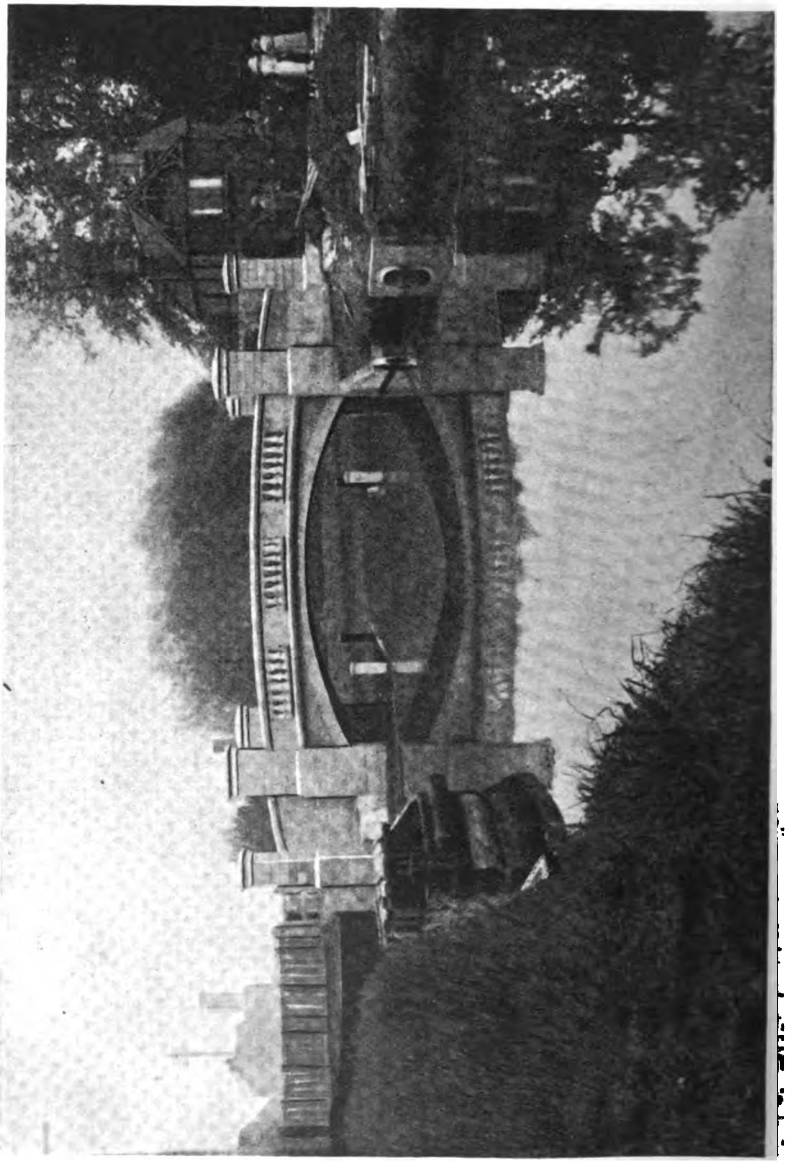


Reinforced Concrete Bridge at Abercynon.

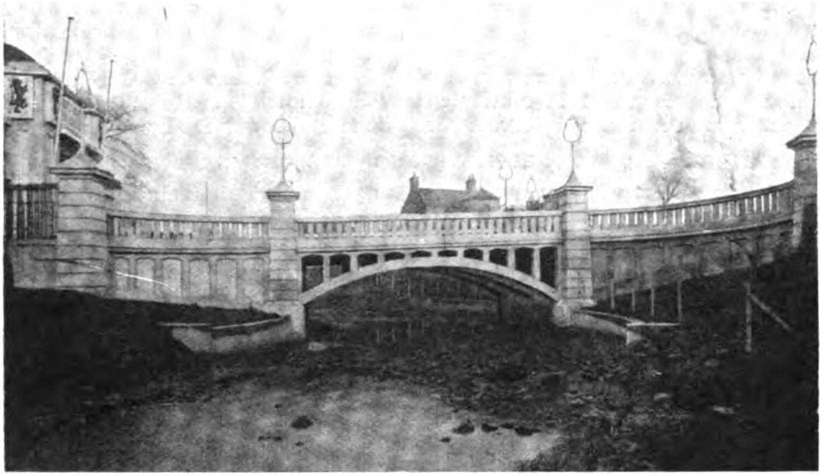
between parapets; arch rib with open spandrils carrying railway tracks. Red Bridge, Ilford: Span, 40 ft.; width 60 ft.; arch vault—single span over river Roding. Northwich footbridge: Span, 28 ft. 7 in.; width, 6 ft. between parapets; parapet girder footbridge; single span over railways. Monkland Canal Bridge: Span, 25 ft. 1 in.; width, 70 ft. between parapets; beam bridge; single span over canal. Gosport Bridge: Spans, centre 35 ft., end 25 ft. 3 in.; width, 30 ft. between parapets; beam bridge; 3 spans over Alver Creek. River Cart Bridge: Span, 78 ft. 9 in. clear; width, 60 ft. between parapets; 2-hinged arch vault with ribs; single span over river Cart. Kinclune Bridge: Span, 51 ft.; cantilevers, 14 ft. 3½ in.; width, 10 ft. 6 in. between parapets; parapet girder bridge, 2 spans, over river Don. Skelton Yard Footbridge: Span, 41 ft.; width, 6 ft. between parapets; parapet girder footbridge; 5 spans, with approach stairs. Gilfach Bridge: Spans, 26 ft. 11 in. and 26 ft. 6½ in.; width, 50 ft. between parapets; beam bridge; 3 spans over railway. Clyde Bridge (Oswald Street), Glasgow: Spans, centre, 166 ft., 2 sides 120 ft.; width, 80 ft. between parapets; arched beam type; 3 spans on circular caissons (illustrated in our July issue). Needham Market Bridge: Span, 43 ft.; width, 14 ft. 6



Reinforced Concrete Bridge at Whittlesey.

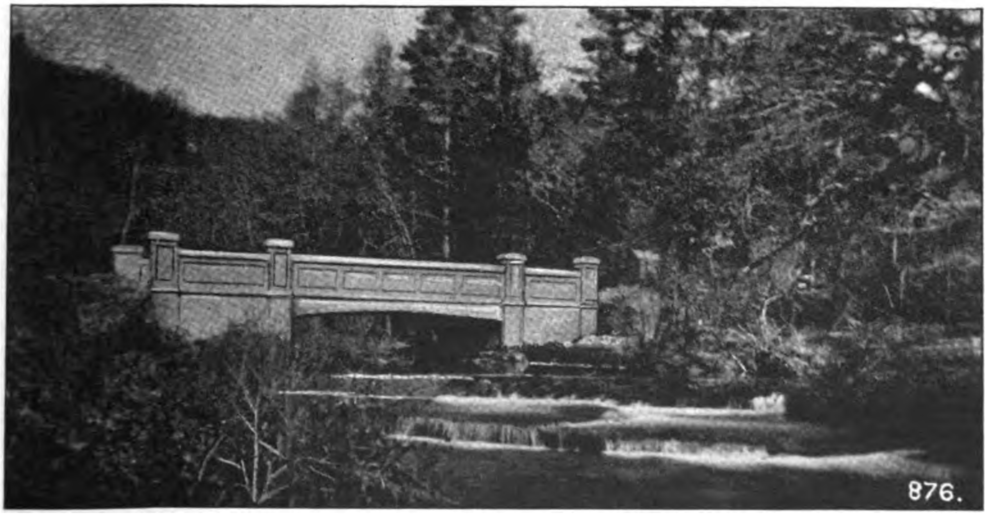


REINFORCED CONCRETE BRIDGE AT ROYDON, ESSEX.



Reinforced Concrete Bridge at Gorgie Road, Edinburgh.

in. between parapets; beam bridge; 3 spans over river. Clyde Navigation East Bridge: Average spans, 31 ft.; width, 80 ft. between parapets; beam bridge; 4 spans over railway. Clyde Navigation Centre Bridge: Spans, 28 ft. and 32 ft.; width, 80 ft. between parapets; beam bridge; 2 spans over railway. Clyde Navigation West Bridge: Spans, 30 ft. and 32 ft.; width 80 ft. between parapets; beam bridge; 4 spans over railway. Pandy Bridge: Span 70 ft.; width, 36 ft. between parapets; open spandril arched bridge over river Ebbw.



Reinforced Concrete Bridge at Doublebois, Cornwall.

CONCRETE SEWERS AND CONDUITS.

The increased adoption of concrete sewers in this country has led to several inquiries reaching us as to the latest American practice. Illustrated articles describing some recently laid reinforced concrete sewers in America, by Professor E. R. Matthews, A.M.Inst.C.E., F.R.S. (Ed.), were given in our issues for April and June, 1922, and we have now received the American Concrete Institute's "Recommended Rules for Monolithic Concrete Sewer Design," as follows:

RULES FOR MONOLITHIC CONCRETE SEWER DESIGN.

(1) Concrete sewers without reinforcement are approved for sizes between 30 and 60 in. mean diameter. Plain concrete sewers between these sizes are to be used only in rock or hard soils. It is recommended that the minimum thickness for a diameter of 36 in. or under should be 5 in. and for a 5 ft. diameter 7 in., with intermediate sizes in proportion. These thicknesses are to be taken as a minimum for circular sewers and used only under favourable conditions.

(2) All sewers near the surface and subject to moving loads or vibration should be reinforced. For sewers of 6 ft. or less in diameter it is recommended that the reinforcement be ½ of 1 per cent., placed near the inside at the crown and near the outside at the springing lines. If it appears at all possible that the horizontal pressures on the sewer might be large, reinforce for reverse stresses.

(3) It is recommended that for all sewers greater than 6 ft. in diameter several possible types of loading be assumed and stresses be calculated on the elastic arch theory.

(4) It is also suggested that in sewers of greater than 6 ft. in diameter it may be found economical to adopt a section having a comparatively flat bottom, and an arch with or without intermediate side walls.

(5) The minimum thickness of concrete in sections of this type should be 8 in. This is recommended as a factor of safety against poor placing and also to secure waterproof structures.

(6) It is recommended that all arches be built of 1 : 2 : 4 concrete and that the inverts be of the same concrete except in rock or very hard soils, where 1 : 2½ : 5 concrete may be used.

(7) For reinforced work in bad ground the designer should provide for a raft of 1 : 3 : 6 concrete of from 4 to 6 in. in depth, which is to be allowed to set before the reinforced structure is started. This is advisable to facilitate good workmanship and particularly to prevent contamination of the concrete around the reinforcement by mud or sand.

(8) The distance from the face of reinforcing steel to the face of the concrete in monolithic sewers should be not less than 2 in.

(9) In determining dimensions of concrete and reinforcement, the following working stresses should be the maximum used:

(*a*) The maximum working stress in the steel where structural grade is used should be not more than 12,000 lb. and for intermediate or hard grades, or for cold twisted bars, 16,000 lb. per sq. in. The maximum working stress in rail steel should not exceed 16,000 lb. per sq. in.

(*b*) Working Stresses in lb. per sq. in.

Aggregate.	Concrete.		
	1 : 2 : 4	1 : 2½ : 5	1 : 3 : 6
Granite or trap rock	550	450	350
Gravel or hard limestone	500	400	325
Soft limestone or sandstone (if permitted)	375	300	250

A 1 : 2½ : 5 concrete is not recommended for use in the sewer proper. Soft limestone and sandstone are prohibited.

These stresses should be further reduced where construction conditions are likely to be very unfavourable to good workmanship, as in very wet or deep trenches.

(10) In all important work the reinforcement should be held in place with steel chairs or holders and wire ties.

(11) Attention is called to the fact that with sewers having comparatively flat inverts careful consideration must be given to the distribution of load across the invert. Where soils are likely to be compressible the weight should be taken as uniformly distributed. The stresses in such inverts should be care-

fully analysed, as they are generally more severe than the other parts of the sewer.

It will generally be advisable to provide alternative details of the invert for use in rock cuts when resting on rock or nearly incompressible soils and for soft or wet ground.

(12) Specifications for monolithic work should provide for either a granolithic finish on the invert or for a lining of concrete block, brick, or tile. The use of the separate lining should be considered as an additional factor of safety where unsatisfactory construction conditions are likely to affect adversely the quality of workmanship and the strength or density of the finished invert concrete.

SIZES AND DIMENSIONS OF CONCRETE SEWER PIPES.

The table on page 736 giving recommended dimensions for pipes made by hand or machine in a concrete works and supplied as a finished product is taken from the specification of the American Society for Testing Materials.

DESIGN AND CONSTRUCTION OF MONOLITHIC SEWERS.

An appendix to the "Proposed Standard Specification of the American Concrete Institute for Monolithic Concrete Sewers," issued in January this year, contains the following :—

CIRCULAR SECTION.

The popularity of this form over others is largely due to the fact that it encloses a given area with the least perimeter, thus affording the highest velocity when flowing half-full or full. It also offers economy in masonry, under favourable circumstances, but where the trench bottoms are flat or where piling or timber platforms are necessary this does not hold true, as the arch requires additional support. In combined systems, however, this section is not as advantageous as the egg-shape, as here the dry weather flow of sewage is small in comparison with the storm-weather flow, and the velocity for the low flow would be less in the circular section. For sewers under 5 ft. in diameter the circular and the egg-shape sections are usually preferred to other types.

EGG-SHAPE.

In the case of combined sewers the egg-shape section is preferred by some from the fact that as the depth of flow decreases the hydraulic radius remains more nearly constant. This advantage also obtains where there is a sanitary sewer to be constructed in a district of which the population served is small in proportion to that anticipated. With the egg-shape section for a given quantity of flow the depth is greater and the velocity slightly higher than would be the case with a circular section. This higher velocity also produces better flotation for solid matter, which in the circular section sometimes tends to obstruct the flow.

A further advantage of this section is that for the same capacity its horizontal diameter is less than in the circular form, which makes possible its construction in a narrower trench. This, of course, in the case of deep trenches, means a saving in excavation. On the other hand, it must be remembered that the vertical height is greater than in the circular type. It is more difficult to construct owing to lack of stability and liability to crack, and in yielding soil it requires more masonry at the haunches to support the arch. In good stiff soil or in rock this may not be necessary. In general the egg-shape section, while probably as near the ideal from the hydraulic standpoint as any yet devised for variable flows, is somewhat more expensive in general than the circular, and far more so than some of the other types to be described.

CATENARY SECTION.

This is primarily advantageous in that it conforms so nearly to the available space inside the wooden timbering in earth tunnels (*see Fig.* 1). It has also the advantage of strength, its line of resistance lying well within the arch section. Its hydraulic properties are fairly good, and the fact that the centre of gravity of the wetted area lies much lower with respect to the height than in the case of the circular section results in the possibility of making lateral connections at a lower elevation, or of raising the invert of the main sewer in cases

TABLE III.—DIMENSIONS OF CEMENT-CONCRETE SEWER PIPE.

Internal Diameter (D), in.	Laying Length (L), ft.	Inside Diameter at Mouth of Socket (D_s), in.[1]	Depth of Socket (L_s), in.	Minimum Taper of Socket (H).	Thickness of Barrel (T, in.	Thickness of Socket (T_s).
4	2, $2\frac{1}{2}$	6	$1\frac{1}{2}$	1 : 20	$\frac{9}{16}$	The thickness of the socket $\frac{1}{2}$ in. from its outer end shall be not less than three fourths of the thickness of the barrel of the pipe.
6	2, $2\frac{1}{2}$	$8\frac{1}{4}$	2	1 : 20	$\frac{5}{8}$	
8	2, $2\frac{1}{2}$, 3	$10\frac{3}{4}$	$2\frac{1}{4}$	1 : 20	$\frac{3}{4}$	
10	2, $2\frac{1}{2}$, 3	13	$2\frac{1}{2}$	1 : 20	$\frac{7}{8}$	
12	2, $2\frac{1}{2}$, 3	$15\frac{1}{4}$	$2\frac{1}{2}$	1 : 20	1	
15	2, $2\frac{1}{2}$, 3	$18\frac{3}{4}$	$2\frac{1}{2}$	1 : 20	$1\frac{1}{4}$	
18	2, $2\frac{1}{2}$, 3	$22\frac{1}{4}$	$2\frac{3}{4}$	1 : 20	$1\frac{1}{2}$	
21	2, $2\frac{1}{2}$, 3	26	$2\frac{3}{4}$	1 : 20	$1\frac{3}{4}$	
24	2, $2\frac{1}{2}$, 3	$29\frac{1}{2}$	3	1 : 20	$2\frac{1}{8}$	
27	3	$33\frac{1}{4}$	$3\frac{1}{4}$	1 : 20	$2\frac{1}{4}$	
30	3	37	$3\frac{1}{2}$	1 : 20	$2\frac{1}{2}$	
33	3	$40\frac{1}{4}$	4	1 : 20	$2\frac{3}{4}$	
36	3	44	4	1 : 20	3	
39	3	$47\frac{1}{4}$	4	1 : 20	$3\frac{1}{4}$	
42	3	51	4	1 : 20	$3\frac{1}{2}$	

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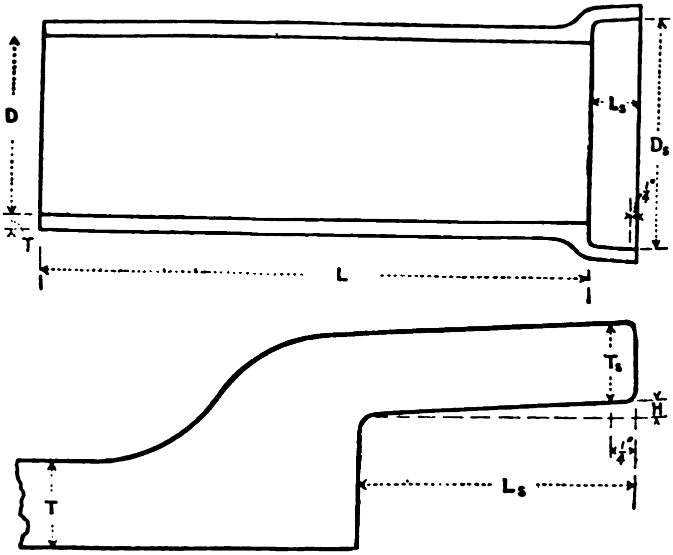
[1] When pipes are furnished having an increase in thickness over that given in last column, then the diameter of socket shall be increased by an amount equal to twice the increase of thickness of barrel.



APPLICATION OF DIMENSIONS GIVEN IN ABOVE TABLE.

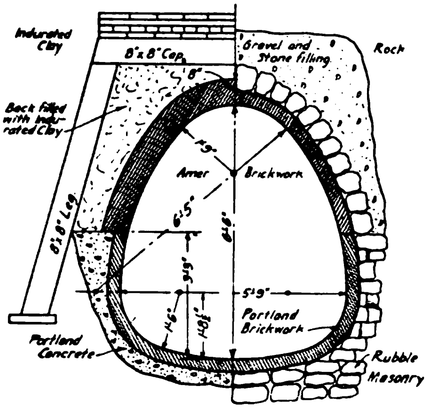


Fig. 1.—Typical Catenary Section.

where it is planned to operate the lateral sewers under a head when the main sewer is running full. In general, however, such a scheme should be regarded with suspicion, though it is certainly an advantage where the allowable difference in water level is small. This type will carry a greater quantity for a given increase in depth than the circular sewer.

This type was used exclusively on the Metropolitan Sewerage Commission work in Massachusetts, serving the suburbs of Boston, in particular on the North Metropolitan sewerage system, but of late years its popularity seems to have waned.

Gothic Section.

Another section (*Fig.* 2), used to some extent on the North Metropolitan sewerages system, is the Gothic, which in shape and advantage closely resembles the circular. Its horizontal diameter is about 17 per cent. less and its vertical diameter about 8 per cent. more than the diameter of the equivalent circle, so that this type requires less width of trench than the circular. Where the sewers are to be located on the basis of the crown grade, however, the greater heights of the Gothic section, necessitating undue excavation, would usually preclude its use. Its hydraulic properties are very similar to those of the circular section and its pointed arch is somewhat stronger than the semi-circular arch. While this type is not in general use at present, it possesses advantages which justify its consideration for special cases.

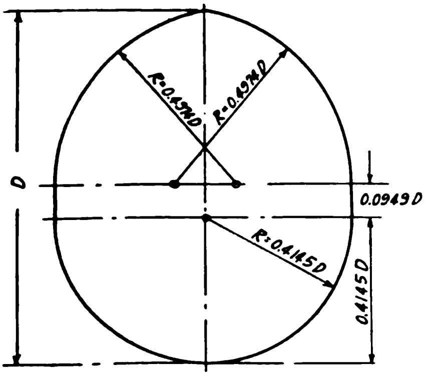


Fig. 2.—Typical Gothic Section.

The Basket-Handle Section.

Fig. 3 is another type extensively used on the Massachusetts North Metropolitan system and also somewhat in other places.

The horizontal diameter is about 6 per cent. less than the vertical. The arch is slightly pointed and the invert is flatter than a semi-circle. In this shape the area, perimeter and the theoretical velocity when flowing more than one-sixth full differ but little from the corresponding elements in a circle having the same height. In actual construction, under the conditions that usually obtain, this shape is more stable, when entirely completed, than a circular shape. It requires more care, however, to prevent injury to the invert while the latter is being constructed.

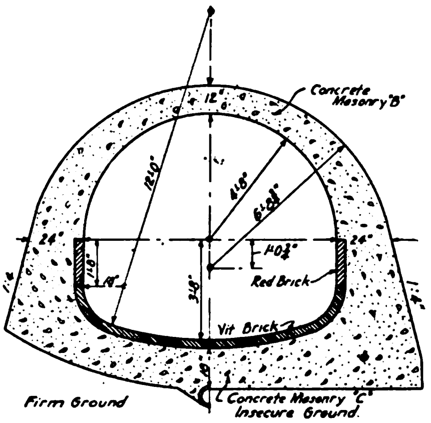


Fig. 3.—Typical Basket-Handle Section.

The basket-handle section closely resembles the horse-shoe type, and has about the same advantages and disadvantages. Its Gothic arch gives slightly greater strength and greater ease in removing the collapsible forms. The large radius curve of its invert and the rounded corners between the side walls and invert may also afford some additional strength, but the horse-shoe type so nearly approximates it in this respect and is so much easier of construction that it is usually used in preference.

HORSE-SHOE SECTION.

The horse-shoe section (*Fig.* 4), next to the circular, is probably the most popular type. While many modifications have been made, in order to adapt it to special conditions, in general it consists of a semi-circular arch upon side walls inclined inward and having either a plane or a curved surface. The invert surface varies in section from a horizontal line to a circular or parabolic arc, and is so constructed as to concentrate the low flow near its centre. In yielding soil it has the advantage over the circular and egg-shape sections of conforming to the trench bottom, so that its arch does not require masonry backing. It may also, for a given width or horizontal diameter, be designed with less height and still carry the same quantity of sewage as the circular section. This decrease in height would result in a saving in excavation in cases where the depth of the sewer below the surface is controlled by the crown grade. Especially where the headroom available is restricted, this wide low horse-shoe type is often advantageous, as it is also in yielding soil where spread foundations are required, as here it affords economy in masonry of the invert. It is possible to broaden out and lower the horse-shoe type until the diminishing side walls disappear and the section becomes semi-circular.

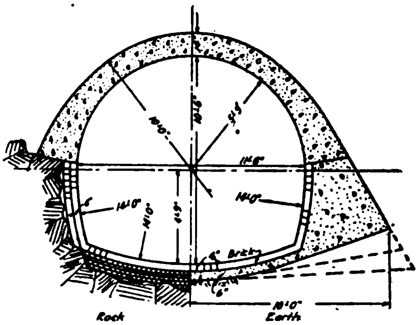


FIG. 4.—TYPICAL HORSE-SHOE SECTION.

Unless this section is built on a very firm foundation, where the invert or base of the side walls will be so supported as to maintain the line of thrust within the section, the whole structure will act as an elastic arch from centre of invert to crown, endangering the stability of the arch. To obviate this lateral thrust it is sometimes necessary in compressible soil to make the side walls extremely heavy. The effect of these side walls is greatly to increase the bending moment at the crown and at the centre of the invert, which will become very severe, especially at the invert centre and at the springing line, if the sewer is constructed monolithic of reinforced concrete with the reinforcing bars continuous from centre of invert to crown of sewer.

In compressible soil heavy side walls or abutments are also necessary. Reinforcement of the concrete has helped to obviate this trouble, but even in the sections heavily reinforced there have been instances of the arch cracking on the inside of the crown and on the outside at the quarter points or near the springing line. Though failure of the arch did not result in these cases, the condition is objectionable as it permits leakage and possible rusting of the reinforcing steel.

SEMI-ELLIPTICAL SECTION

This section (*Fig.* 5) may be either a true semi-ellipse or may be composed of three circular arcs approximating the semi-ellipse. Its chief advantage over the other sections lies in the fact that under working conditions the shape of its arch more nearly coincides with the lines of arch resistance. Hence the arch section can be made relatively thin without creating excessive stresses in the masonry. Unlike the horse-shoe type, this section depends but little on the lateral pressure of the earth, and not at all upon the passive pressure or natural resistance of the earth filling. It also has the advantage over the circular type of a lower normal flow, since the centre of gravity of the wetted area is lower with respect to the crown. Its use is advantageous in earth tunnels where the usual form of timbering is used, since like the catenary this type conforms readily to the available space.

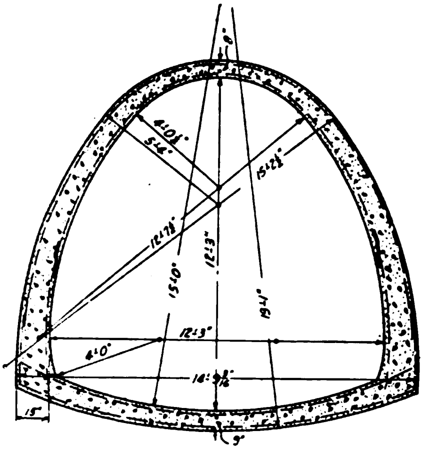


Fig. 5.—Typical Semi-Elliptical Section.

With respect to the invert, in this type its stability is more important than the other types thus far discussed, as the arch is thin in section and extends nearly down to the invert line, making necessary the distribution of the invert pressure over a large area. In cases where the section is constructed in compressible soil and the sewer is monolithic (with steel reinforcing bars running continuously from centre of invert to crown of sewer) the invert should be made as thick as the arch at the springing line, and should be heavily reinforced to resist the large bending moment of the invert centre. Otherwise cracks may occur at this point. The quantity of masonry below the springing line is not excessive in this section, for, like the horse-shoe type, its invert conforms to the trench bottoms.

While the hydraulic properties of this section are in general very good, its wide and shallow invert does not afford as high a velocity during times of low flow as does the circular section. Where the amount of sewage to be provided for is not liable to wide variations and the normal flow is equal to or greater than one-third of the total capacity of the sewer, this disadvantage may be disregarded. In fact, for sewers constructed of reinforced concrete and over 6 ft. in diameter this type is one of the best.

Parabolic, or Delta, Section.

The parabolic, or delta, section shown in *Fig.* 6 was developed by Mr. James H. Fuertes in designing the sewerage system for Santos, Brazil, and in 1902 he used a similar section for the city of Harrisburg, Pennsylvania. This section is nearly triangular, being composed of an arc in the form of a parabola and an invert consisting of a short-circular arc with side slopes of about 3 horizontal to 1 vertical. It may be designed to carry a somewhat greater flow for its height than a circular sewer. In addition to being both economical and strong, its sloping invert gives a lower normal flow line than the circular section, which is of value in districts where the available fall is limited, and also in cities where owing to tidal water the sewers must be built in shallow cut. In low land, also, or where there can be but little depth of excavation, it offers a further advantage because the greater carrying capacity below the springing line afforded by the wide invert makes possible the building of a section of less height than in the case of the circular sewer. This is especially true where the excavation is so shallow that embankment work is necessary.

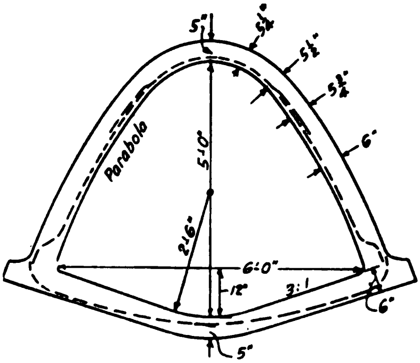


Fig. 6.—Typical Parabolic (or Delta) Section.

This section has the disadvantage over the semi-elliptical type that its pointed arch requires a relatively wider section for equal capacity and height.

Elliptical Section.

There are a few sewers in the United States built with an elliptical section

(*Fig.* 7). This section, both above and below the springing line, approximates a portion of an ellipse, the longer axis being sometimes vertical and sometimes horizontal. In general the elliptical section has so little to commend it that it has not proved popular. It has the same disadvantage as the egg-shape section that unless it is built in very firm soil additional masonry backing will be needed under the haunches, to support the arch.

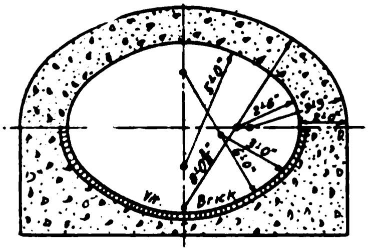


Fig. 7.—Typical Elliptical Section.

U Shape Section.

The U-shape section is used to a limited extent for sewers in the vicinity of 3 ft. in diameter. Undoubtedly it offers some advantage where the width of trench is limited, and where there is sufficient headroom available to build a sewer with the horizontal diameter much less than the vertical. Its hydraulic properties, also, are fairly good, though when it becomes filled the hydraulic mean radius is materially reduced due to the addition of the width of the slab roof to the wetted perimeter. Its pointed shape affords greater ease in the removal of forms than is the case with the circular section, and the invert is of such shape that good velocities are obtained at times of low flow.

Rectangular Section.

The rectangular section (*Fig.* 8) has been used for many years to meet special conditions, as where the headroom or sideroom in the trench was restricted. It is easy of construction, economical of space and of masonry, and has good hydraulic properties up to the point where the flat top becomes wet. Just before this point is reached the velocity and discharge are relatively large, but they decrease rapidly as soon as the wetted perimeter is increased and the hydraulic radius decreased by the wetting of the top. Hence it is usual in designing these sections to provide for an air space of from 3 to 12 in. above the maximum flow line.

This type is of special advantage in deep rock-cuts. The usual form has a less height than width, but the hydraulic properties of this section become less favourable as the ratio of height to width increases. There is an advantage in the narrow high section, as this permits reduction in the width of excavation often sufficient more than to offset the increase in depth of trench.

This type of section requires care in design, in order to ensure its stability. If the top is built in the form of a flat slab it must be designed as a beam to provide for the earth load, and the side walls must be strengthened to resist lateral pressure. If designed as an arch it is necessary to strengthen the side walls to withstand the thrust

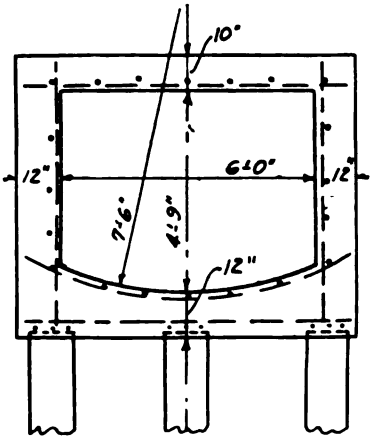


Fig. 8.—Typical Rectangular Section.

The flat slab top may be constructed ether of I-beams encased in concrete or as a concrete slab reinforced with bar. The first method (while not economical of steel as the I-beams must take all the load, the concrete merely acting as a filler) is of advantage in that it is possible to complete the sewer and backfill the trench more quickly than where the roof is built as a reinforced slab. Also, the beams are easily and quickly placed, and it is claimed in certain cases this ease of construction offsets the additional cost. In the case of a large sewer built in a congested district, the advantage of speed, permitting prompt backfilling of the trench, may be an important factor. However, the method of construction requires great care to protect the steel from rust.

Quite frequently the rectangular section is constructed with a V-shape invert to gain greater velocity for low flows.

Semi-Circular Section.

In New York City and vicinity the semi-circular section (*Fig.* 9), though not as popular as the rectangular, is being rather extensively used. Its hydraulic properties are not equal to those of the rectangular section, and it requires a wider trench and more extensive foundations for equal capacity and height than most of the other types described. Its use is most advantageous for outfall sewers crossing low lands, where the natural surface of the ground is largely below the top of the sewer arch or even below the invert. The invert must be firm and designed to resist the thrust of the arch. Consequently this type as a rule requires more masonry in proportion to the capacity afforded than do the other types. In some instances two or more semi-circular sections have been built side by side in order to save headroom.

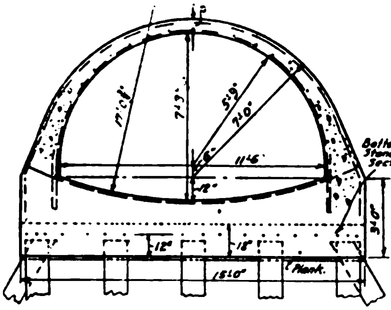


Fig. 9.—Typical Semi-Circular Section.

Double and Triple Sections.

These are especially useful in the case of outfall sewers in thickly settled districts, or where a storm-water sewer is constructed above or below a large sanitary sewer for the sake of economy in excavation or headroom, or to meet other special problems.

The Cunette.

The "Cunette" (*Fig.* 10) is a special dry weather channel sometimes constructed in the invert of various types of sewers. It has not proved especially popular in the United States, but in France and in Germany has been used extensively. It has the advantage of providing a good channel during times of low flow and thus maintaining self-cleansing velocities. It has the disadvantage of requiring greater depth of trench and additional masonry.

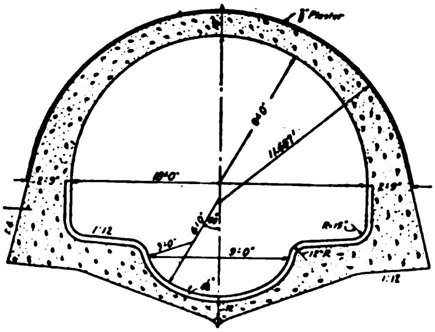


Fig. 10.—Section with a Cunette.

[Many of the illustrations given with this report are taken from "American Sewerage Practice," by the courtesy of the publishers, The McGraw-Hill Book Co.]

GENERAL NOTES ON SEWER CONSTRUCTION.

The report of the American Concrete Institute also contains some interesting notes on the design and construction of concrete sewers from which we take the following :—

Methods of Design.

The inside shape of a concrete sewer is often a compromise between the best hydraulic shape, the best shape for structural strength with economy, a shape which is easy to build, and conditions of height imposed by head room or hydraulic conditions. Up to a diameter of 6 ft. a circular shape will frequently meet these conditions satisfactorily. As the size increases, more attention must be given to the shape for structural strength.

In large sizes with flat inverts (dictated by ease of construction) the intensity of soil pressure at any point under the invert requires careful study to determine the thickness and amount of reinforcement. This also will be a factor in deciding whether the sewer is to be analysed as an elastic ring or the top analysed as an arch with fixed abutments.

The water in the sewer must be taken into account in determining the stresses on the sewer ring. This is especially important with light cover and heavy eccentric loads. Also, conditions during construction or later may occur when the water may exert a bursting pressure and require reinforcement to resist it.

Economy may be shown by revising the trial sewer shape to make the line of resistance for the normal dead load condition fall within the middle third of the arch ring.

It is usual to specify that the sides of the sewer below the springing line and the bottom are to rest against the undisturbed soil or against the sheeting left in place. While this allows the arch to be supported by the passive soil pressure, it is better practice only to calculate on the active soil pressure, as some deflection of the sewer or movement of the soil must occur before the passive pressure comes into action.

The design of the sewer section necessitates a study of the conditions under which, it is to be built and under which it is to serve. Both live and dead load have to be considered carefully. In large construction, with shallow cover, the live load, as from motor lorries, road rollers, tramcars, etc., may be extremely important. In deep cover, the live load is of less importance. Soil conditions also have to be considered. In general the horizontal earth load to be used in the design may vary with soil conditions from one-third the vertical load in a relatively stiff soil to a load in quicksand increasing uniformly with the depth, on a basis of approximately 100 lb. per sq. ft. per ft. depth. This loading is more intense than a water load.

For the surface loadings, allowance must be made according to local conditions. On shallow depth sewers unsymmetrical loadings should be investigated, and this has become particularly important because of the increased weight of traffic. The loads should be considered from the standpoint of the crowded street, which, of course, lessens the reduction made by any allowance for distribution of load through the earth cover.

ANALYSIS OF STRESSES.

On the smaller sections, say up to 6 ft. internal diameter, practice on uniform soil in different localities has demonstrated in part a reasonable section. However, special conditions may necessitate a thorough analysis of the stresses. On the larger sections, say 6½ ft. and upwards, a special analysis of the stresses is desirable, and particularly so with sections 8 ft. in major dimension and larger. Such analysis can be made along the lines of the elastic theory, or the modified method suggested by Metcalf and Eddy may be used except where eccentric loadings have to be investigated. In some cases, the old voussoir method of analysis still has application in the study of lines of stress in the sewer ring. The larger the section the more careful should be the analysis of the stresses and a complete consideration of all details, the condition of the ground, kind of backfilling, with possibility of abnormal stress of unbalanced backfill, and the type of loading to be expected.

When the analysis has been made consideration should be given to future conditions of operation and maintenance. On sewers carrying special industrial wastes or very septic human sewage, or exposed to extremely high velocities, a factor of protection or additional thickness over the theoretical may be well worth while.

STANDARD SECTIONS.

In any given locality, where the soil conditions are reasonably uniform, standard sections and thicknesses may readily be worked up. Such standardised sections may run in size from tile pipe 6-in. in diameter up to sections with a major dimension of say 6 ft. Such standards will also require occasional adjustment, where special sections are required, for hydraulic reasons.

PLAIN *v.* REINFORCED CONSTRUCTION.

The structural properties of concrete are such that for a structure subjected to possible bending, as a sewer may normally be, the basic condition on the elastic arch theory leads to a reinforced shell.

Where direct compression combines with the bending moments in a way to eliminate tensile stresses, a condition of zero reinforcement occurs. It is probably true that many extreme designs of concrete and of brick sewers are standing after incipient failure because of the passive resistance of the earth at the haunches. This makes necessary a close scrutiny of the character of the soil, whether hard or soft, in drawing up a design.

In hard material with sufficient cover the shape of the sewer arch can often be arranged to conform to the load line found by analysis. This may save reinforcing. Rectangular sewers will always require reinforcing in the top beam, unless this is so thick that arch action occurs, when it may not be economical. The sides and bottom will usually need reinfórcing also, particularly in the softer materials.

As a general rule in the smaller sizes, from 3 ft. up to 8 ft. in diameter, contractors prefer to build monolithic concrete sewers without reinforcement. This makes for simplicity and speed in construction, as the concrete can usually be deposited more quickly than the steel can be placed. Plain monolithic concrete is most suitable in trenches of moderate depth, in firm soils, with light live loads, in locations where no restrictions are placed on head room. While monolithic sewers have been built in firm soils up to 17 ft. in diameter without reinforcement, bottom conditions or special loadings may dictate a small amount of steel as insurance. Certainly in large sewers from 10 ft. upwards steel reinforcement is often desirable and generally used. The cross-section of concrete is thereby reduced. Reinforced concrete sewers are structurally applicable to all conditions of soil and loading. An analysis of cost under these particular conditions may show the best course to pursue. In the design sometimes cases occur where the invert alone may need reinforcement, and again where the arch alone may need it.

In the field the use of reinforcing steel to meet unexpected soil conditions may make local use in any sewer worth while. In soft soils the sewer must act as an elastic ring rather than an arch, in order to distribute the load over the foundation. This condition will usually require some reinforcement except in the very small sizes.

REINFORCEMENT.

In sewer construction of any size, bar reinforcement is used rather than mesh. Except in very small sewers, the mesh does not supply enough steel to meet the needs of the designer. This is also because of the amount of steel required in one direction across the arch. Only sufficient longitudinal steel is required for spacing and shrinkage or temperature stresses during construction. The amount of longitudinal steel varies with the designer and the conditions; in some offices as low as 0·1 of 1 per cent. of the cross-section has been used. For tight work free from cracks as high as 0·4 of 1 per cent. is used.

INFLUENCE OF SOIL CONDITIONS.

Soil conditions are one of the most important points in the design of a sewer. While soil conditions may be learned approximately by borings and preliminary exploration, the true soil condition is often not learned until every foot is opened up. While soil conditions materially influence the design of the invert in most types of sewers, they have a most important bearing on all arch design, unless the arch is made self-contained, as was done in the early days in brick sewer design. Consequently a knowledge of soil conditions is all-important.

In hard stiff soils, where complete and stiff backing can be obtained with concrete, a thinner section can be used than in loose running soils or compressible material. In all soils care should be taken to have the backfill thoroughly compacted on the haunches, unless the sewer is so designed as to minimise the value of this precaution.

In rock a sewer invert may be called upon to take but little stress. In stiff soils the invert will distribute readily the load from the sides without making heavy stresses. But in compressible soils the invert may require special design and reinforcing to distribute evenly the weight of the sides and the bending movements on the soil material without cracking. For this purpose and in emergencies reinforcing steel is very useful.

In very soft foundations, where thickening of the invert or reinforcing with steel will not suffice, plank foundations alone, or plank on piling, may be required. The extent and amount of timbering or piling depend on the local conditions and should be governed by experience. Plank foundations are often made with 3 in. plank laid on the trench bottom, in two layers crossed. Where piling is required to bridge sand pockets, mud or similar material, the piles are driven first. On the pile bents waling stays may be spiked, on which are laid longitudinally planks to sustain the concrete until thoroughly hardened.

In very soft soil conditions during construction may call for foundation piling or planking that will not be required after the sewer is completed and the trench backfilled, as the completed sewer full of water will not differ by a large percentage from the weight of the soil displaced. The empty sewer may even show a tendency to rise or float. This leaves the foundations under the sewer with the sole use of preventing unequal settlement. With longitudinal reinforcement the sewer, especially the side walls, becomes a continuous girder that will bridge over small soft spots.

INVERT LINING.

Sewer inverts are usually built of concrete in concrete sewers. In the past, where excessive velocities have been expected, lining with tile or brick has been adopted. Except in extreme cases this is not resorted to in present practice so much as in the past. Where steep grades and grit are encountered this protective phase should be investigated. Lining means additional cost, because the use of a material of this type as a lining may slow up construction. Rich dense concrete floated to a hard finish should be good for velocities up to 15 or even 20 ft. per second. In a sewer such velocities are usually only very occasional.

The resistance of concrete or other materials to scouring action is a subject on which little data is available. Tests indicate that good concrete wears surprisingly well, and that under continued wear of heavy abrasion in tests 6 in. of good concrete should be equivalent under the same conditions to a high grade paving brick 4 in. in thickness.

In placing concrete on inverts the method of construction must be carefully worked out to secure the best hydraulic results, i.e., a smooth invert on uniform grade. A liberal use of pieces of reinforcing steel driven in to correct grade is helpful. In some shapes of sewer the use of pre-cast concrete blocks to carry the forms has proved worth while. In other shapes, the placing of a segment of the bottom in a section has been successfully screeded to a hard finish. In large sections with comparatively flat bottoms the bottom should be placed first and preferably screeded to finish. Plastering afterwards on a rough bottom is difficult.

PLACING CONCRETE.

The consistency should be plastic enough to flow readily in the forms and around reinforcing steel. According to modern investigations, too wet a mix is as bad as too dry. The sidewalls and lower portion of the arch can be put in with a wetter mix than the crown. By judicious use of outside forms and choice of consistency a dense arch can be secured. Where trenches are wet, the mix for the bottom is often made rather dry to absorb moisture in the trench. Where the invert is reinforced a lean mix may be placed as a skin coat in moist trenches.

REMOVAL OF FORMS.

The hardening of the concrete before removal of forms varies considerably with the weather and size of the sewer. In warm weather removal is possible in 24 to 36 hours, on the smaller sizes, and 48 hours and upwards on the larger. In cold weather longer periods are desirable. Backfilling usually can commence in from 3 to 7 days. Care should be taken to avoid eccentric loading and shock from dumping.

BOOK REVIEWS.

Handbook of Construction Cost. By H. P. Gillette.
(London: McGraw-Hill Book Co. 1734 pp. + xvii. Price 30s. net.)

This volume is a new companion book to the "Handbook of Cost Data," first published about seventeen years ago, and which was prepared by the author for the purpose of dealing with costs in a manner somewhat different from that usually adopted because the data presented was in the form of units of material and hours of labour rather than as actual money values. The present volume carries the subject of costs another stage forward, because the author has deduced formulæ for commodity price levels and wage levels which will hold good at all times, and it is only necessary to obtain average values for the variable factors at the time of using the formulæ in order to arrive at the cost required.

The greater part of the information in the book will not be applicable in this country, as it relates to work in America, but the principles of costing will be the same, and the methods explained are worthy of study. The quantity surveyor is practically unknown in America, where estimating and costing are usually carried out in a very general manner and without the strict attention to detail that is associated with qualified quantity surveyors over here. It is important to realise this fact when reading some portions of the book, otherwise the reader will not appreciate the author's comments. As an example, he points out many items which the American contractor frequently overlooks when preparing an estimate, and when it is mentioned that these items include insurance, temporary buildings, water supply, protection of work executed, cutting and jobbing for other trades, and similar matters which are taken as a matter of course when estimating in this country, this will be realised.

The author has compiled a great amount of data covering practically every trade and aspect of construction work, and the complete nature of the book should prove a distinct asset and make the volume valuable when dealing with the costs of work in America, while the methods explained and adopted by the author make the book a useful one to all those interested in construction costs.—A.L.

Elements of Graphic Statics. By Hudson and Squire.
(London: McGraw-Hill Publishing Co., Ltd. Price 6s. 3*d*. net.)

The authors of this little book (87 pages, crown 8vo) are on the professorial staff of the Polytechnic Institute of Brooklyn, New York, and their purpose is to present the essentials of graphic statics, especially in their application to the calculations of reactions, stresses, shears and moments.

After an introductory chapter on general considerations describing the usual method of representing and combining forces graphically, there is a chapter on reactions which expresses the subject clearly and rather fully. Then follows a chapter on stresses in simple structures, the authors apparently regarding all simple structures as framed structures. The fourth and last chapter deals with moment and shear, and includes an investigation of the masonry arch, although many simpler problems more suitable to the beginner are not dealt with.

The book does not appear to differ materially from many of the numerous books previously published upon the subject. We notice a tendency—rather common with American writers—of using one term to describe a given construction without mentioning that other terms are in use, perhaps more common use. We note, for instance, that the "equilibrium polygon" is explained without saying that other writers (and we believe most of the earlier writers) call this the "link polygon" or the "funicular polygon." In the treatment of stresses in simple structures the method of joints is described, but the authors do not say that this is the same as the method of reciprocal figures, although we believe the latter term is in most common use. The authors tell us that points at which the members of a truss are united are called "joints," "apexes" (dreadful word!) or "panel points," but do not appear to have heard of "nodes."

The book contains some problems for the student to work, but problems without answers are not of much value to students working privately; they can set themselves problems, but they want to know if their results are correct.—E. S. A.

REINFORCED
CONCRETE
PIT-HEADS.
(*See p.* 747.)

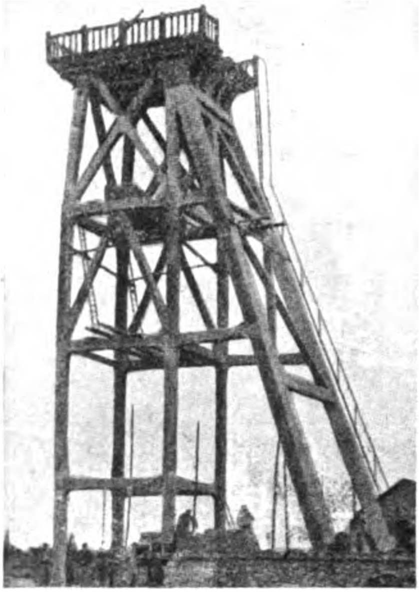


FIG. 3. CONCRETE PIT-HEAD NEAR LENS.

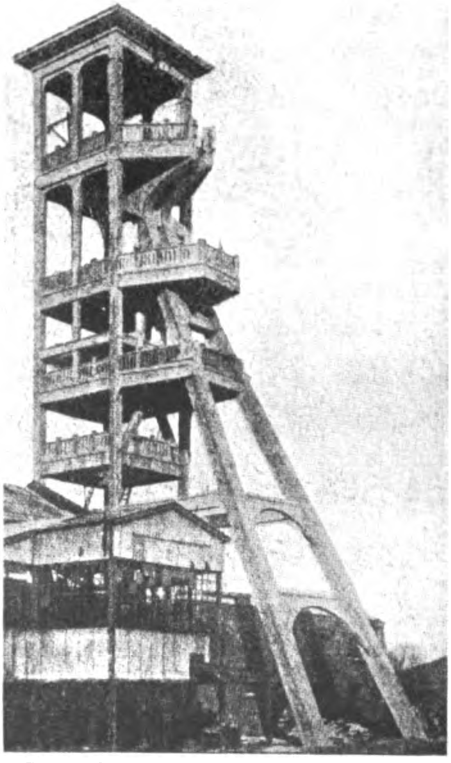


FIG. 2.—CONCRETE PIT-HEAD AT THE ANRZIN MINE.

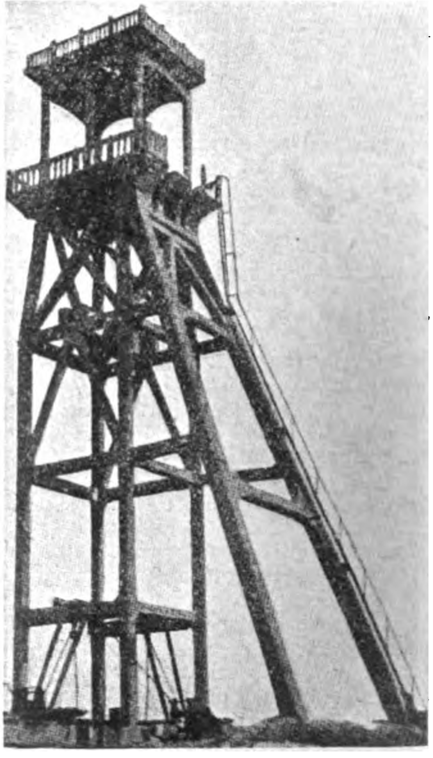


FIG. 4.—CONCRETE PIT-HEAD NEAR LENS.

PIT-HEADS OF REINFORCED CONCRETE.

WHEN the winding engine is contained in the pit-head itself, so that the lift is wholly vertical, the pit-head may be in the form of a self-contained tower, but if, as is more usual, the engine is in a separate building, so that there is a tension in an oblique direction, this moment must be resisted by means of stays or shores.

A pit-head of the first type, 160 ft. high, and with a ground area of 46 ft. by 33 ft., is shown in *Fig.* 1. Another of the same type, but only 90 ft. high, built in 1914, is shown in *Fig.* 2.

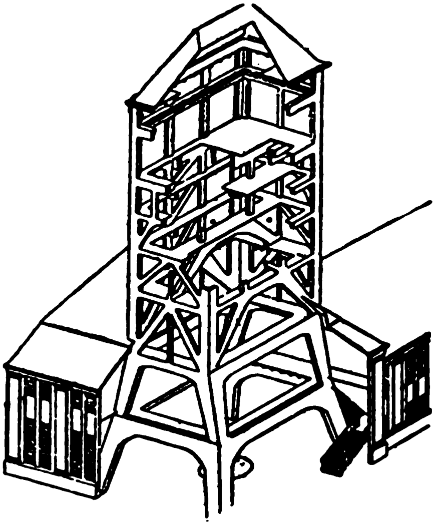


FIG. 1. CONCRETE PIT-HEAD.

Pit-heads with stays or shores are shown in *Figs.* 3 and 4, both these being 79 ft. high, or 65 ft. to the pulleys. It is essential that the upper part of the structure carrying the pulleys shall be of ample strength. A gangway is also desirable, in order to facilitate frequent inspection of this portion of the machinery. A roof over the pulleys protects them from the weather, and is also useful in fixing them in position or in removing them when required.

The stays should be able to resist a pressure of 200 tons, or to the breaking strain of the rope if that exceeds this figure. The structure is subject to a considerable amount of vibration, and, though this is less serious with concrete than with steel structures, provision must be made for it.

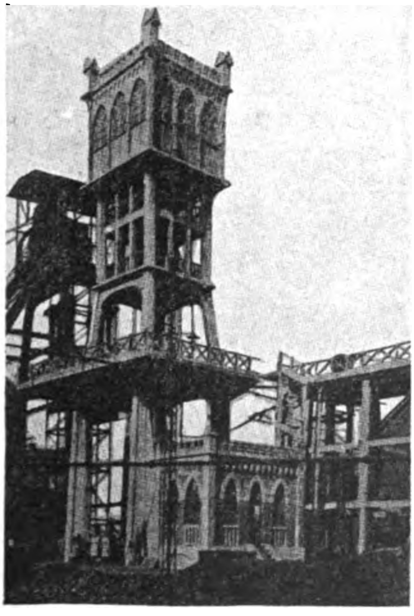


FIG. 5.—PIT-HEAD AT LA HAZE, NEAR LIÈGE.

The illustrations show pit-heads constructed in 1912–1914 which, notwithstanding continuous use, show no cracks or other signs of fatigue. In some cases concrete pit-heads have been subjected to shocks equal to those produced by the breaking of the winding-rope, but they have not been in any way damaged thereby.

No difficulties have arisen as a result of such heavy structures being placed immediately above the shaft, as was at one time feared might be the case. No repairs have yet been necessary, but when they are required it will not be difficult to make them quickly and cheaply.

The time required to construct the largest concrete pit-heads does not exceed six months, including the foundations, whereas the time required for steel construction is much greater. The cost of construction varies with the ease with which the raw materials can be obtained, but is usually about half that of a steel structure.—*Revue du Beton Armé.*

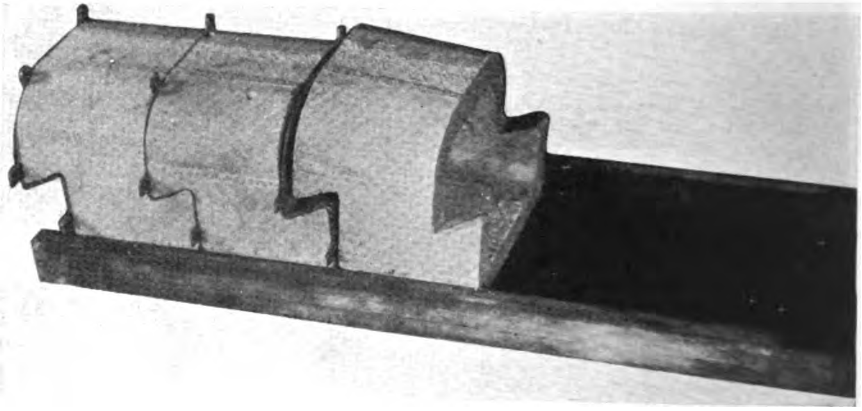


FIG. 2.—SECTION OF INTERLOCKING CULVERT CONSTRUCTION.

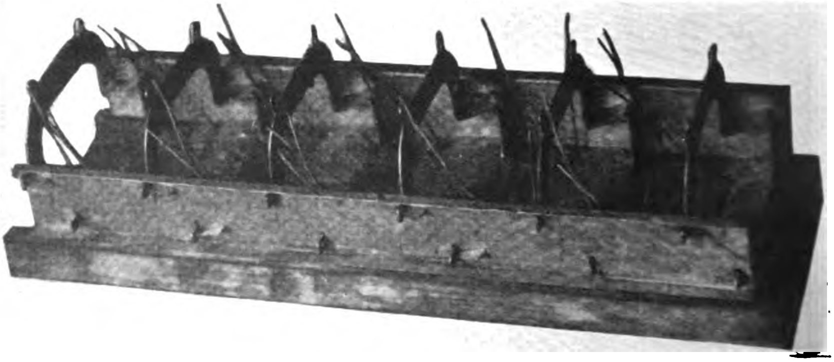


FIG. I.—FORM FOR INTERLOCKING CONCRETE CULVERT SECTIONS.

AMERICAN REINFORCED CONCRETE CULVERTS. (*See p.* 749.)

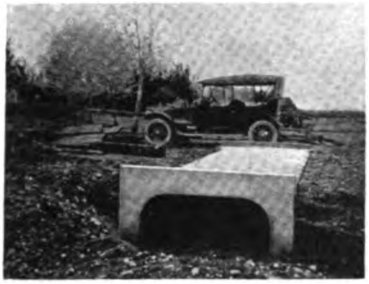


FIG. 3.

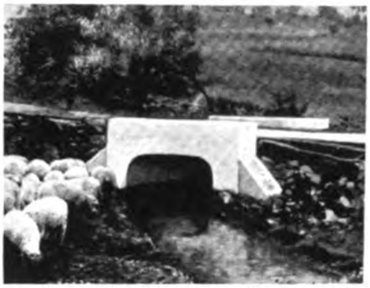


FIG. 5.

AMERICAN REINFORCED CONCRETE CULVERTS.

THE accompanying illustrations, *Figs.* 1 and 2, show the "Hall" interlocking concrete culvert developed at Chicago, Ill., with the sections under construction and the forms. It is claimed that this type of culvert is economical as it eliminates waste of materials by unskilled labour, and formwork on the site. With the use of interlocking concrete culverts traffic is not interfered with, as the sections can be transported from storage to the site, and the work completed in a single day. The necessity for making temporary roads is eliminated, as well as the time required in making and curing poured concrete.

Safety is secured by end-blocks which lock in with other blocks. Five sections are made at one casting.

It is claimed that the interlocking feature guarantees the unity of the finished culvert, while at the same time providing expansion joints. It is elastic in construction and can be lengthened for a 30-ft. road, or a 24-in. diameter culvert can be replaced by one of 36-in. or more as needed without waste.

Fig. 3 shows a 24-ft. concrete culvert constructed at Helena, Montana, utilising in its construction the "Whalen" concrete form developed at Syracuse, N.Y., shown in *Fig.* 4. This form is constructed on the unit method of construction; the width of all form sections is 24 in., the length 72 in., and the total height as erected 26 in. The side walls are 15 in. high, and the arched tops are also 15 in. high, having an average radius of 12 in. The arched tops are placed into the side walls in such a manner that they rest on the top set of cross rods for the side walls, making the height of a section 26 in.

Fig. 5 shows a concrete culvert built on this system with a span of 48 in., a height of 26 in., and length of 24 ft.

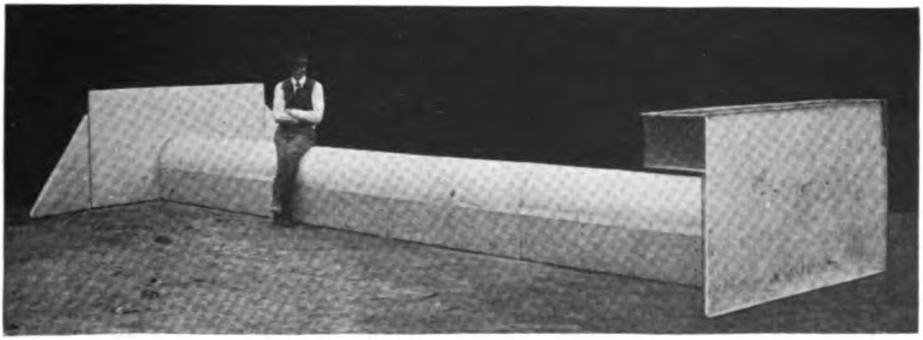
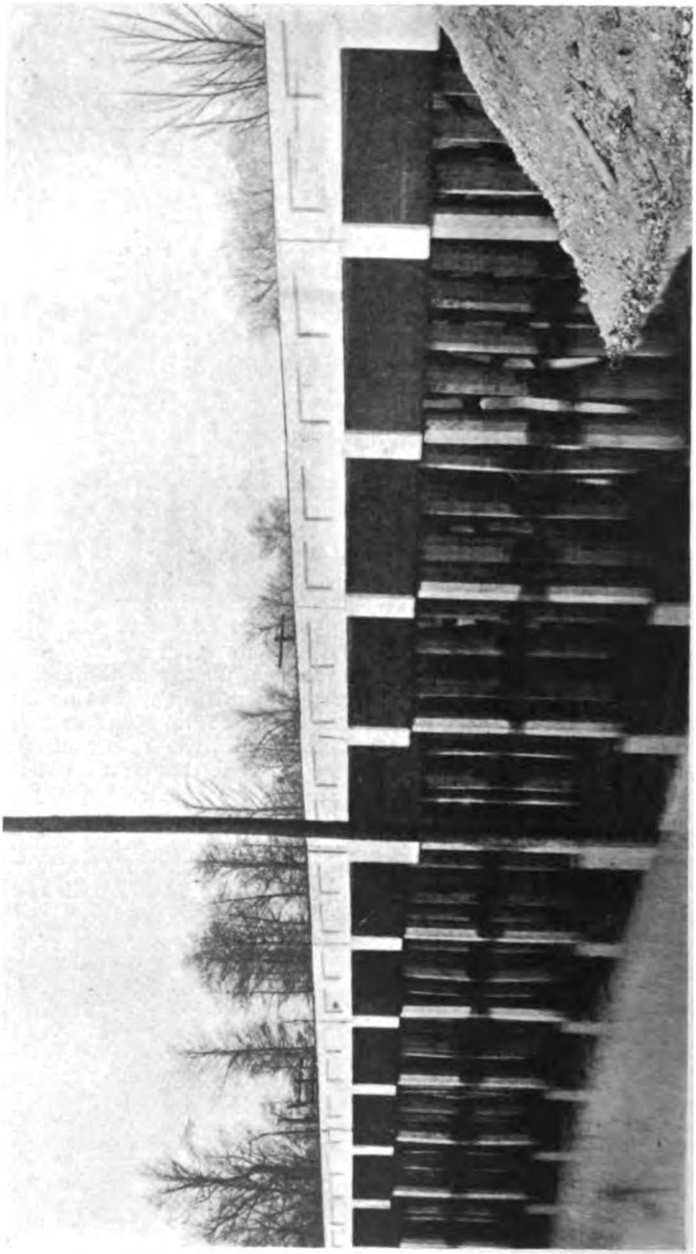


FIG. 4.



CONCRETE BRIDGE ON THE ILLINOIS CENTRAL RAILROAD. (See p. 751.)

RAILROAD CONCRETE PILE CONSTRUCTION.

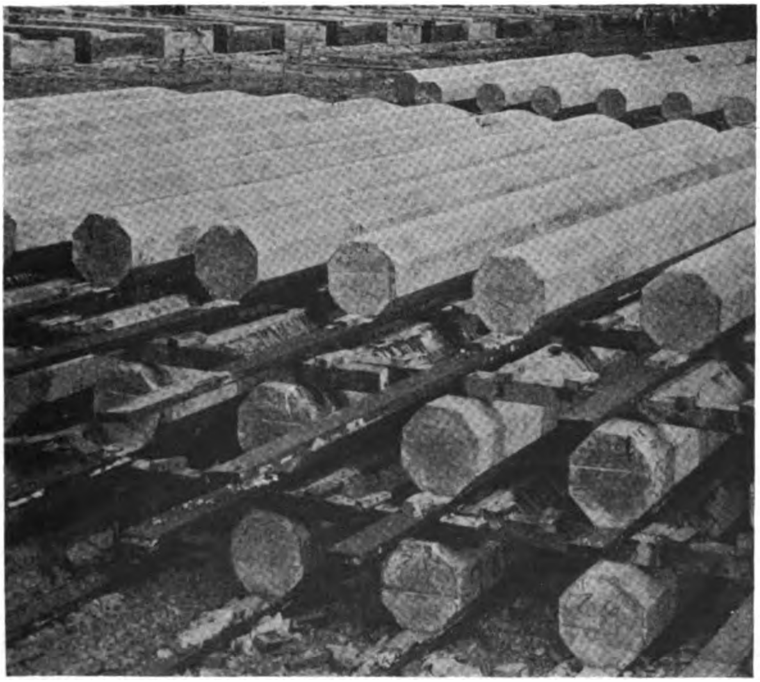
THE accompanying illustrations show a concrete pile yard at Memphis, Tennessee, and the concrete pile bridge of the Illinois Central Railroad.

Concrete piles are used by the American railroads for a variety of purposes, among which are foundations for buildings, bridges, viaducts, retaining walls, conduits, station platforms, docks, wharves, turntables, cinder pits, and reinforced concrete slab trestle bridges.

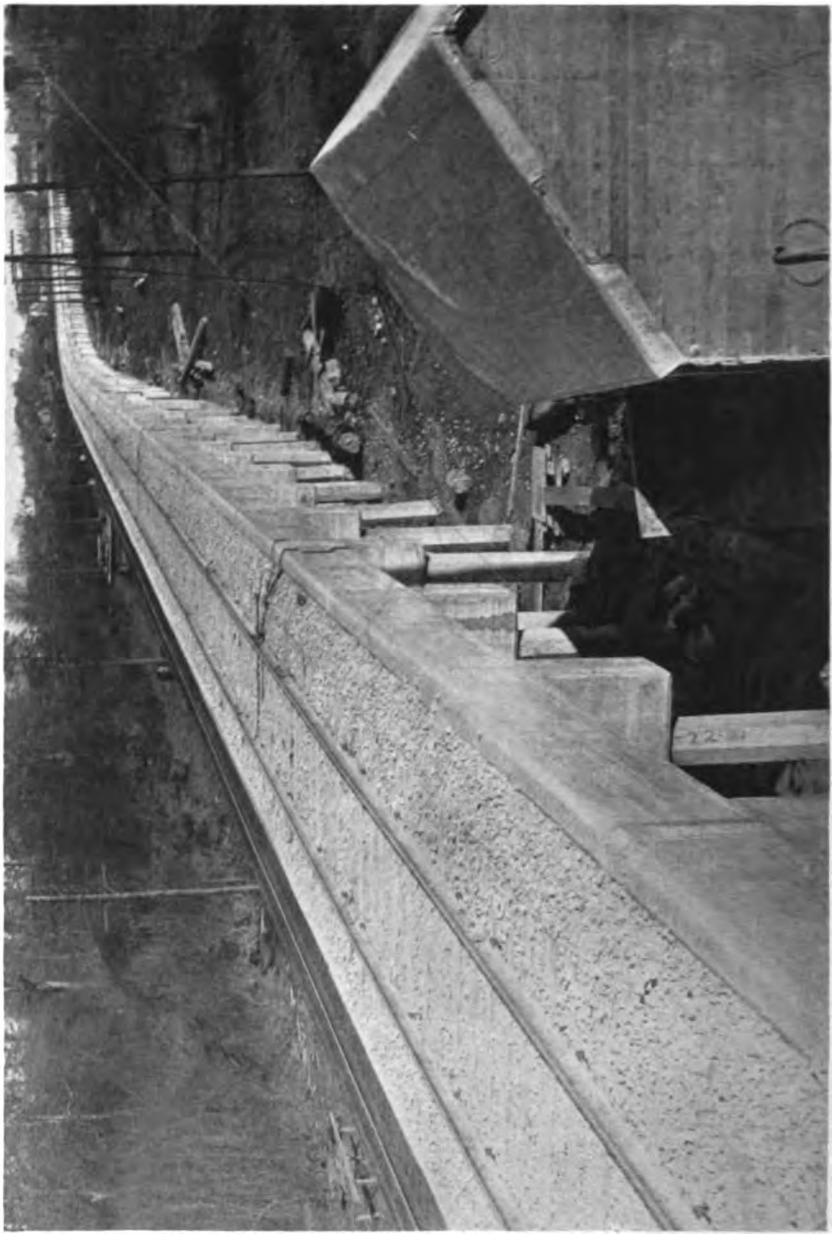
The Chicago, Burlington and Quincy Railway, which was a pioneer in this type of construction, has constructed about 20 miles of concrete pile trestles, some of which have been in service eight years. The Chicago, Milwaukee and St. Paul Railway started manufacturing and using concrete piles for concrete trestles in 1912, and since that time upwards of 30,000 lin. ft. have been used. The Great Northern, the Illinois Central, the Minneapolis, St. Paul and Sault Ste. Marie, the Northern Pacific and the Wheeling and Lake Erie Railways are among the roads which make considerable use of concrete piles for trestles. The Great Northern, however, now prefers slim reinforced concrete piers extending 1 to 2 ft. below the surface of the ground, and supported on piles, with longitudinal struts for heights about 20 ft.

It is generally agreed in the United States that pre-moulded reinforced concrete piles are suitable for use in trestle bents; they allow loads of from 20 to 35 tons per pile and the penetration required under ordinary conditions varies from about two-fifths to two-thirds the length of the pile, while the maximum projection above the ground recommended varies from 10 ft. to 30 ft. Several railways fix about 20 ft. for this maximum projection above ground unsupported, while some limit this height to about 14 ft. or 16 ft. and build thin reinforced concrete piers for greater heights.

The pre-moulded type of pile is preferred by many railways owing to the necessity for using this type for trestle work. The octagonal, straight-sided pile about 16 in. in diameter appears to be the



CONCRETE PILE YARD AT MEMPHIS.



BALLAST-DECK CONCRETE PILE TRESTLE ON THE ILLINOIS CENTRAL RAILROAD.

most generally used. The steel reinforcement of concrete piles is designed not only to take a portion of the load that may be placed upon the pile after it is driven, but also to take care of the bending stresses that occur when the pile is lifted either by the middle or by one end, to withstand the shocks caused in dragging it over rough ground to the site where it is to be used and the jars occasioned in driving.

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

SILICATE OF SODA AND CONCRETE.

FURTHER to the Question in our Sept. issue, a correspondent writes as follows:—

" The treatment of concrete surfaces with silicate of soda gives beneficial results at any time regardless of the age of the concrete. In treating a newly-laid floor, the treatment should preferably be carried out from a week to a fortnight after laying. It is extremely important that the floor should be dry and that the silicate of soda should be diluted so as to ensure maximum penetration. Treatment of old floors which have already started to crumble is, however, to be recommended. The tests carried out by the U.S. Bureau of Standards to which reference has already been made in this Journal (page 449, 1923) were carried out upon floors which had been laid and used at least five months before treatment. For old concrete floors it is customary to use a more alkaline silicate than that usually recommended for the treatment of new floors."

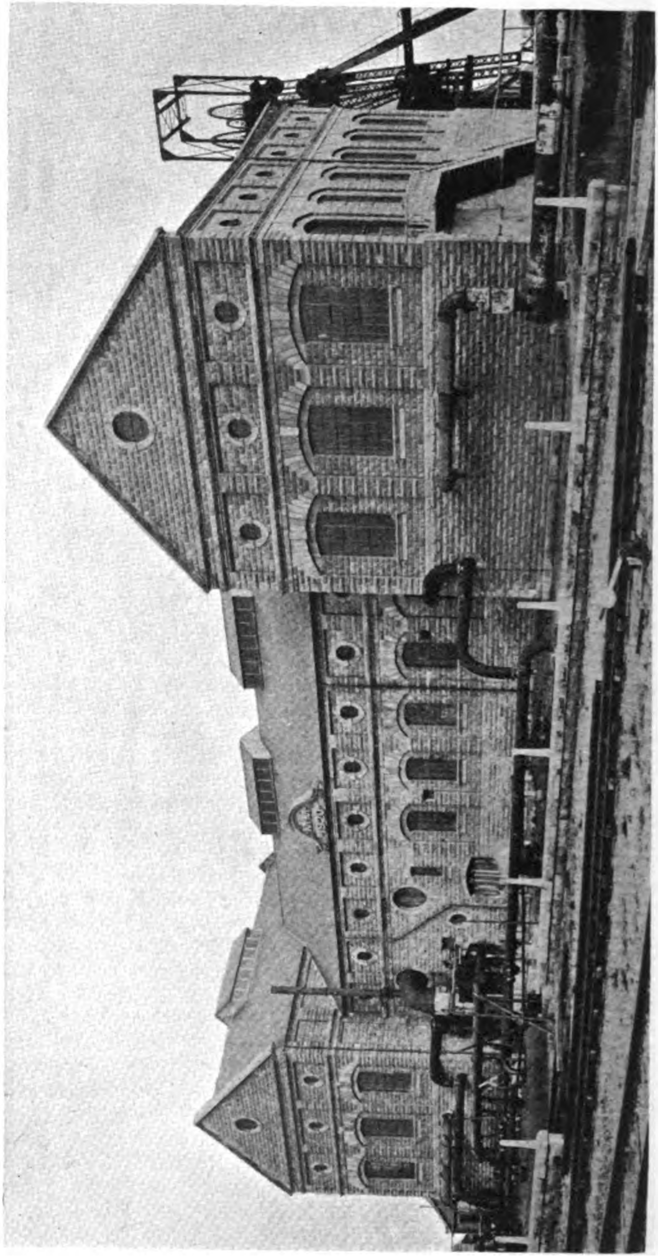
" THE CONCRETE YEAR BOOK, 1924."

THE large number of inquiries we are continually receiving for names of makers of different specialities, addresses of firms and engineers, etc., have convinced us that there is a great need for a complete directory of the many branches of the concrete industry. It is, in fact, an anomaly that a growing industry which has already reached such large proportions should be without its directory, and we have decided to fill this gap by the publication early in January next of the first edition of " The Concrete Year Book." This volume will be a Handbook and Directory for the Concrete Industry, and should be of considerable value to all interested in the many branches of that industry.

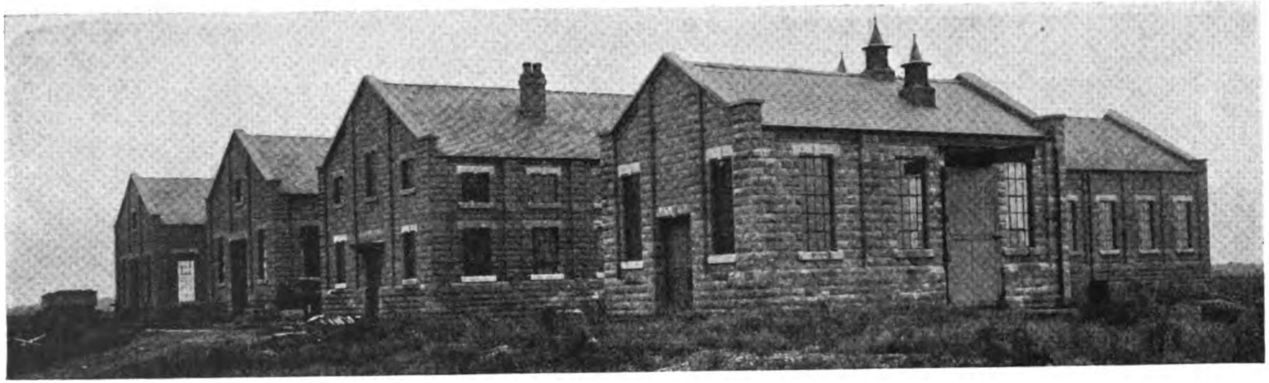
The first part of the book will consist of special articles dealing with such subjects as the economical use of cement, essentials in the production of good concrete, proportioning and grading of aggregates, concrete floors, concrete roads, concreting in cold weather, and other aspects of the uses of concrete. A specially-prepared series of tables for reinforced concrete design will also be included. Another feature will be a bibliography of all books dealing with reinforced concrete and allied subjects.

The remainder of the book will contain a complete Directory of the industry. classified under different headings, including Consulting Engineers who Specialise in Reinforced Concrete, Reinforced Concrete Designing Firms, Public Works Contractors who Specialise in Concrete Construction, Concrete Flooring Specialists, Suppliers of Aggregates and Sand, Artificial Stone Manufacturers, Manufacturers of Colouring Materials for Concrete, Manufacturers of Machinery and Plant for Concrete and General Constructional Work, Manufacturers of Concrete Products, Manufacturers of Reinforcement, Contractors for Concrete Roads, Manufacturers of Proofers and Hardeners for Concrete, Piling Contractors, Manufacturers of Asbestos-Cement Products, Concrete Testers, etc. No pains have been spared to make the Directory Section as complete and accurate as possible.

The volume, which will be strongly bound, will be published by Concrete Publications, Ltd., Publishing Department, 4 Catherine Street, Aldwych, W.C.2, price 3*s*. post free, and copies may now be ordered.



CONCRETE BUILDINGS AT MARKHAM COLLIERY.



[Mr. Philip M. Brundell, Architect.

CONCRETE BUILDINGS AT MARKHAM COLLIERY.

SOME of the possibilities of concrete block construction for colliery buildings are shown in these two photographs, illustrating some of the concrete buildings recently completed for the Markham Main Colliery, near Doncaster. The buildings were erected by Messrs. Henry Boot & Sons (London), Ltd., from designs by Mr. Philip M. Brundell, architect, now at Harworth Colliery, Bawtry, Yorkshire. One of the reasons which influenced the late Sir Arthur Markham, head of the Markham Colliery Company, to have these buildings constructed in concrete blocks was that the site was a gravel one, and there was therefore no difficulty in obtaining the necessary aggregate.

CONCRETE IN SEA WATER.

THE Committee of the Institution of Civil Engineers on The Deterioration of Structures in Sea Water has now issued its third (interim) report. This report is mainly devoted to researches on the destruction of timber in sea water. From the reports dealing with concrete we take the following :—

Mr. E. A. Cullen, M.Inst.C.E., Engineer to the Harbours and Rivers Department of Brisbane, Queensland, referring to some test blocks of reinforced concrete placed in position in the Brisbane River in November, 1917, states that the eight blocks which were prepared to represent different thicknesses of cover over the steel reinforcement are immersed in salt water, and since the last examination, which took place 18 months ago, the increased extent of the deterioration is rather less than was expected. The rusting has progressed slightly, and in one instance, in block No. 1, the expansion due to this action has been sufficient to cause the concrete to split off. The cracks noticed in blocks 1 and 2 have opened slightly. There are a few new rust stains additional to those found in 1920, but those in block No. 5 have increased and one small crack has developed in No. 6 since the last report.

Blocks Nos. 1 and 2, with only ½ in. of cover, showed early signs of rusting ; Nos. 3 and 4, with 1 in. of cover, showed slight signs of rusting by March, 1919, with considerable increase subsequently. Nos. 5 and 6, with 1½ in. of cover, showed no stains at March, 1919 ; No. 5 showed one rust spot by August, 1920, No. 6 being then without fault, and in March, 1922, both Nos. 5 and 6 showed slight rusting, No. 5 in the shape of two rust pits about 1 in. in diameter, and No. 6 one small crack about 6 in. long, about 2 in. from the edge. Nos. 7 and 8, with 2 in. of cover, are at present without sign of fault.

Comparing the quantity of cement used, in block No. 1, which has a mortar of 2 sand and 1 cement, with that used in block No. 2, with a mixture of 4 sand and 3 cement, both are in bad condition, but No. 1 is the worse of the two, whilst block No. 4, with a mortar of 4 sand and 3 cement, is somewhat worse than No. 3, which has a 2 to 1 mortar. Both blocks, however, are deteriorating very noticeably. Block No. 5, with a 2 to 1 mortar, is slightly worse than No. 6 with a 4 to 3 mortar, but from a deterioration point of view the damage is negligible. These two specimens have only been injured to a very slight extent.

So far as the experiment goes, it seems to show that there is little difference, if any, in respect of deterioration between mortars of 2 sand and 1 cement and those composed of 4 sand to 3 cement.

An account of the present condition of a small reinforced concrete wharf built by Mr. Cullen in the Brisbane River, completed in March, 1917, is also given.

The dimensions of the wharf are 100 ft. by 42 ft., on piles 15 in. by 15 in. The piles and girders have 2 in. of cover over the steel reinforcement, the deck slab has 1¼ in. cover on the under surface, and 1 in. cover at the top. The structure was well tarred with hot distilled tar one month after completion, and again a year later, the first coat having appreciably become very thin. Now, after 5½ years have expired, no deterioration from rusting of the steel can be seen on close examination. The wharf has, however, not yet been subjected to much strain, as it has been little used. The tarring was applied to all surfaces but that of the deck, and was carried down the piles to about the level of mean neap high water. Below mean sea level the piles are encrusted with small barnacles. This result indicates that with scrupulous care in construction, the cover, as above described, and with the surface well tarred at least twice, no corrosion has occurred in a structure in sea-water for at least 5½ years, and possibly no corrosion will take place at all.

Reinforced Concrete Pier at Hove.

A NEW pier, which will include a residential club with forty bedrooms, a theatre to accommodate 1,400, bandstand, kursaal, dance-hall, and public restaurant, is to be built immediately at Hove. The whole of the structure will be carried out in reinforced concrete, at an estimated cost of £80,000.

DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS.

		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	„	14	0
„ „ ½ in. mesh	„	15	0
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Portland Cement	10s. per ton extra		

Boarding for Shuttering—

		Sawn. s.	Sawn. d.	Wrot. s.	Wrot. d.
1 in.	per square	23	6	27	6
1¼ in.	„	29	6	33	6
1½ in.	„	35	6	41	0

Sawn Timber for Struts and Supports—

3 in. by 4 in.	from £24 per standard
3 in. by 6 in. and 3 in. by 7 in.	„ £25 „ „

Mild Steel Rods for Reinforcement—

		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
⁷⁄₁₆ in. to ½ in. Rounds	„	13	6
⅜ in. Rounds	„	14	6
¼ in. Rounds	„	15	6

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

Portland Cement Concrete well mixed with Clean Water in proportion 1 : 2 : 4—

				s.	d.
Do.	do.	in foundation	per yard cube	45	0
Do.	do.	in columns	„ „	50	0
Do.	do.	in beams	„ „	50	0
Do.	do.	in floor slabs 4 in. thick	per yard super	5	3
Do.	do.	in floor slabs 6 in. thick	„ „	7	9
Do.	do.	in floor slabs 9 in. thick	„ „	11	3
Do.	do.	in walls 6 in. thick	„ „	8	0

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

Steel Reinforcement, including Cutting, Bending and putting into Position and securing with Stout Binding Wire—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	23	6
„ ⁷⁄₁₆ in. to ½ in.	„	22	6
„ ⅝ in. to 2⅞ in.	„	21	6

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2¼*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.*

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.*

Shuttering—

		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	54	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	50	0
Do. do. in small quantities	per ft. super	0	9
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	10
Do. do. as last, in narrow widths	„ „	1	0
Do. do. to sides and soffits of beams average 9 in. by 12 in.	„ „	1	0
Do. do. as last, in narrow widths	„ „	1	2
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	„ „	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

WAGES.—The rates of wages on which the above prices are based are those current in the London area, namely:—Carpenters and joiners, 1/7½ per hour; Labourers on building works, 1/2¾; Operators on concrete mixers and hoists, 1/3¾.

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

MEMORANDA.

PROSPECTIVE NEW CONCRETE WORK.

Aberystwyth.—*Sewage Works.*—The T.C. has applied to the Ministry of Health for sanction to borrow £23,006 in connection with a scheme of sewerage and sewage disposal works.

Barnet.—*Houses.*—The U.D.C. will spend £20,000 on housing schemes.

Brighton.—*Houses.*—The T.C. propose erecting 185 dwellings under the Government's housing scheme.

Bristol.—*Dock.*—The City Council has adopted the scheme for the extension of the Bristol Corporation docks at Avonmouth, estimated to cost £1,000,000.

Brownhills.—*Houses.*—The U.D.C. proposes to erect 100 houses.

Cardiff.—*Sea Wall.*—The Council is to apply to the Government for financial assistance to build a sea wall at Splott foreshore.

Carlisle.—*Houses.*—The T.C. has agreed to erect 100 dwellings.

Catford (London).—*Bridge.*—The L.C.C. proposes to erect a bridge over the River Pool at Catford.

Chelmsford.—*Reservoir.*—The T.C. is negotiating for the purchase of land for the construction of a reservoir.

Chester.—*Houses.*—The Corporation will erect forty-nine cottages on the Heath Lane site.

Chipping Ongar.—*Sewage Works.*—The R.D.C. has applied to the Ministry of Health for sanction to borrow £1,000 for sewage disposal works at Hallsford Bridge.

Cleadon.—*Road.*—The R.D.C. is considering a proposal for the construction of a new road to Cleadon, estimated to cost £32,000.

Coatbridge.—*Houses.*—The T.C. proposes to build 200 houses at Whifflet.

Colwyn Bay.—*Houses.*—The U.D.C. is to erect between 200 and 300 houses.

Dunbar.—*Sea Wall.*—The T.C. is considering a scheme for the construction of a sea wall at Belhaven.

East Dean.—*New Road.*—The R.D.C. will submit a scheme for the construction of a new ferro-concrete road, estimated to cost £150,000, to the M.T. for approval.

Filey.—*Waterworks.*—The U.D.C. has adopted a scheme for extending the water supply, which includes the construction of a ferro-concrete covered tank estimated to cost £2,000.

Glasgow.—*Bridge.*—A scheme has been recommended to the Corporation for the construction of a new street and bridge over the River Kelvin at Queen Margaret College estimated to cost £120,500.

Gosport.—*Esplanade.*—The T.C. will submit to the Ministry of Health a relief scheme for the construction of a concrete esplanade, estimated to cost £3,200.

Guildford.—*Houses.*—The Ministry of Health has granted permission to the T.C. for the erection of fifty houses on the Guildford Park site.

Hinckley.—*Sewage Works.*—The U.D.C. is to apply to the Ministry of Health for sanction to borrow £34,400 for new sewage disposal works.

Horsforth.—*Road and Bridge.*—The new Calverley road and bridge scheme, estimated to cost £137,000, has been approved by the M.T. and the West Riding C.C.

Hove.—*Sea Wall.*—The T.C. proposes to extend the sea wall.

Huddersfield.—*Tanks.*—The Dalton Chemical Works Co., Ltd., has decided to erect new benzol storage tanks at its works.

Hull.—*Houses.*—The Corporation is to erect 600 houses on the three Corporation housing estates.

Jarrow.—*Houses.*—The Council has adopted a scheme for the erection of 100 houses.

Jedburgh.—*Bridge.*—A scheme for the reconstruction in reinforced concrete of Jedburgh Station Bridge, at an estimated cost of £5,810, has been approved of by the T.C.

Leigh-on-Sea.—*Sewage Works.*—The Southend-on-Sea Corporation has received permission from the Ministry of Health to borrow £16,500 for the improvement of the Leigh sewage disposal works.

Market Bosworth.—*Sewage Works.*—The R.D.C. has applied to the Ministry of Health for sanction to borrow £6,670 for sewage disposal works at Ibstock.

Nelson.—*Gasholder.*—The Corporation has applied to the Board of Trade for the acquisition of 6½ acres of land at Brierfield as a site for an additional gasholder.

Northampton.—*Houses.*—The T.C. has agreed to erect 234 dwellings.

Oldham.—*Houses.*—The Corporation proposes to build 100 houses.

Penyard.—*Culvert.*—The Merthyr T.C. will apply to the Ministry of Health for sanction to borrow £2,000 for completing works at the Waterloo Bridge Culvert, Penyard.

Plymouth.—*Houses.*—The Corporation contemplates building 1,030 houses and 40 shops.

Poole.—*Reservoir.*—The Corporation proposes constructing a new storage reservoir at Corfe Hills with a capacity of 5,000,000 gallons.

St. Helens.—*Reservoir.*—A scheme for a new reservoir, estimated to cost £100,000, has been decided upon by the Corporation.

Seaton.—*Esplanade.*—The U.D.C. proposes to erect a new esplanade, estimated to cost £3,500.

SOUTH SHIELDS.—*Houses.*—The Corporation proposes to erect 600 houses.

SUTTON.—*Reservoir.*—The Keighley R.D.C. will apply to the Ministry of Health for sanction to borrow £9,400 for the construction of a new reservoir at Sutton-in-Craven.

SWANSEA.—*Station.*—The construction of a new G.W.R. station in High Street, estimated to cost £300,000, is to be put in hand shortly.

TAUNTON.—*Sewage Works.*—The T.C. has applied to the Ministry of Health for sanction to borrow £29,000 for sewage disposal works.

VENTNOR.—*Pier Works.*—The U.D.C. will spend about £1,200 on pier improvements.

WORCESTER.—*Concrete Work.*—It is proposed to pave about six acres of land at the sewage works with a layer of concrete for sludge drying purposes, estimated to cost £5,000.

TENDERS ACCEPTED.

BATH.—*Foundations.*—The T.C. has accepted the tender of Messrs. Hayward and Wooster, at £234, for the construction of a machinery foundation at the generating station.

BATH.—*Reservoir.*—The T.C. has accepted the tender of Messrs. Hayward and Wooster, at £449 15*s.*, for the construction of a reinforced concrete reservoir at Weston.

BEDWORTH.—*Sewage Works.*—The tender of Messrs. G. Percy Trentham & Co., Ltd., at £6,919 10*s.* 3*d.*, for sewage works, has been recommended for acceptance.

CHELSFIELD.—*Reservoir.*—The Metropolitan Water Board has accepted the tender of Messrs. Fred and T. Thorne, at £6,804 13*s.* 2*d.* for the construction of a covered service reservoir.

DUBLIN.—*Roads.*—The tender of Mr. J. Kenny, at £6,167, for the construction of reinforced concrete carriageways, has been recommended for acceptance.

ERPINGHAM.—*Sea Wall.*—The R.D.C. has accepted the tender of Messrs. J. Youngs & Son, Norwich, at £1,717, for the construction of a sea wall at Overstrand.

'GLASGOW.—*Bridges.*—The Clyde Navigation Trustees have accepted the tender of Messrs. Gray's Ferro-Concrete Co., Glasgow, at £25,491, for the construction of the east and centre bridges in connection with the diversion of Renfrew Road at Shieldhall.

HULL.—*Power House.*—The T.C. has accepted the tender of Messrs. Quibell & Son, at £19,587, for ferro-concrete work, etc., at the power house.

RAWRETH, Essex.—*Culverts.*—The Essex C.C. has accepted the tender of Messrs. W. and C. French, Buckhurst Hill, at £1,922 7*s.*, for the construction of three culverts at Rawreth Wash, the diversion of the stream and erection of fences.

SLOUGH.—*Kerb.*—The U.D.C. has accepted the tender of Messrs. W. Wood & Sons, Ltd., at £56 6*s.* 1*d.*, for supplying and fixing concrete kerbing at the new pleasure ground.

STOCKPORT.—*Sewage Works Extensions.*—The Sewage Outfall Committee has accepted the tender of Messrs. P. D. Hayes & Sons, Stockport, at £22,116, for the construction of precipitation tanks and other appurtenant works at the Sewage Works, Cheadle Heath.

TIVERTON.—*Bath.*—The T.C. has accepted the tender of Mr. W. H. Smith, Torquay, at £2,050, for the construction of a new concrete bath at Loughborough.

WALTHAM ABBEY.—*Wall.*—The U.D.C. has accepted the tender of Mr. P. R. Paul, at £219, for the construction of a concrete retaining wall at Cobbins Brook, Galley Hill Road.

URMSTON, LANCS.—*Flagging.*—The U.D.C. has accepted the tender of Messrs. G. Clarke & Sons, Manchester, at £575 6*s.* 8*d.*, for supplying and laying about 1,200 sq. yds. of concrete flagging in Church Road.

WORCESTER.—*Bridge.*—The Corporation has accepted the tender of Messrs. T. Vale & Sons, Ltd., Stourport, at £9,255 9*s.* 6*d.*, for the reconstruction of Bransford bridge in ferro-concrete.

RECENT PATENT APPLICATIONS.

181,713.—G. M. Gouyaud : Reinforced concrete arches.

199,758.—M. Haramiishi : Ferro-concrete structures.

200,276.—F. C. C. Rings and Anglo-Danish Concrete Construction Co., Ltd. : Blockwork floors, roofs, etc.

200,311.—J. Sperni : Manufacture of Magnesian cement.

200,419.—J. McWhirter : Concrete or other cast slabs or blocks.

200,564.—F. Hartgrove : Concrete block waterproof dressing.

201,098.—A. Henderson : Concrete building blocks.

201,509.—M. Haramiishi : Ferro-concrete floors and walls.

201,650.—Soc. Anon Prodor and Dr. M. Levy : Manufacture of concrete.

201,867.—A. W. Dickinson : Combined hollow-tile and reinforced concrete floors and ceilings.

202,129.—J. G. Dudley : Cement-coated metal-framed buildings.

202,404.—British Thomson-Houston Co., Ltd. (General Electric Co.), Cement impervious to the action of water.

202,572.—A. Henderson: Concrete beams, girders and columns.

202,742.—P. C. Cannon and Centrifugal Concrete Blocks & Poles, Ltd.: Manufacture of concrete products.

202,856.—S. W. Mobbs: Groynes or walls for coast protection.

203,055.—H. S. Tudsbery: Apparatus for testing the physical qualities of materials.

203,088.—R. G. Kirton: Mould-board apparatus for concrete structures.

203,112.—A. S. Douglass: Building blocks.

203,132.—British Everite and Asbestilite Works, Ltd., and E. F. Jucker: Roofing tiles or sheets.

203,208.—G. J. Hone and J. Westwood & Co., Ltd.: Grabs.

203,414.—W. Savage and Ruston & Hornsby, Ltd.: Excavating machines.

203,528.—D. McLean: Ventilation of cavity walls and building structures.

204,224.—G. Jaeger: Water tanks for concrete mixing machines.

NEW COMPANIES REGISTERED.

Bradshaw's Reinforced Concrete Co. Ltd. (192013). Windsor House, Victoria Street, S.W.1. Registered August 21. Engineers and concrete specialists. Nominal capital £2,000 in 2,000 shares of £1 each. Directors: F. W. Bradshaw, 3 Nursery Villas, Hitchin; and W. J. Webb, Windermere High Road, Loughton. Qualification of directors, 1 share; Remuneration of directors to be voted by company.

Hydra Lime Products, Ltd. (191598). Registered July 28. 25 Cross Street, Manchester. Manufacturers or merchants of lime, mortar, cement, etc. Nominal capital £3,000 in 2,700 ordinary shares of £1 each and 6,000 deferred ordinary 1s. shares. Directors: T. S. Smith, 25 Cross Street, Manchester; R. Sharples, 22 Fortuna Grove, Levenshulme, Manchester; and F. Howett, Middleton Road West, Chadderton, Oldham. Qualification of directors, £1; remuneration to be voted by company.

TRADE NOTICES.

Under the title "B.R.C. Structures," the British Reinforced Concrete Engineering Co., Ltd., of Manchester, have issued a volume of over 360 pages illustrating buildings and other structures in which the firm's reinforcement has been used. The works illustrated cover practically the whole range of structures in which reinforced concrete is used, including factory buildings and warehouses for various purposes, railway sheds, sidings, station platforms, garages, boiler houses, aeroplane sheds, sawmills, weaving sheds, oil tanks, docks, slipways, mills, pottery ovens, pumping stations, gas works, crane tracks, housing schemes, office buildings, cement works, coal stores, timber sheds, shop premises, showrooms, banks, power stations, electricity generating stations, schools, swimming baths, residential buildings, etc. Although the majority of the illustrations are of buildings in this country, many large buildings abroad are included. One page is devoted to each building or work under construction, and the photographs are well reproduced. The views of the Lady Lever Art Gallery, erected at Port Sunlight for Viscount Leverhulme, are the most complete set we have seen of this building, and should be of considerable interest to architects. The book is clearly printed on good paper, and neatly bound in stiff boards.

The film exhibition, "Concrete Roads," arranged by the British Reinforced Concrete Engineering Co. Ltd., of Manchester, which depicts the various operations necessary in the construction of complete concrete roads, was given at the Royal Cinema, Belfast, last month. The scheme of roadmaking comprised in the film, seven miles in extent, was at Tilbury Marshes, where the subsoil was alluvium or river mud. All the processes in the construction were shown, and it was pointed out that cross-overs were provided for mains to enable repairs to be made where pipes crossed under the roads without breaking up the main slab.

Messrs. Brunner, Mond & Co., Ltd., of Northwich, have issued a pamphlet entitled "Silicate of Soda and Concrete," which contains information on the properties and action of silicate of soda in hardening concrete surfaces.

Messrs. Wm. Boyer & Sons, of Irongate Wharf, Paddington Basin, W., supplied all the aggregate, amounting to over 5,000 tons, for the construction of the grain silo at Avonmouth for the Co-operative Wholesale Society, illustrated in our last issue.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

DECEMBER 1923 VOL. XVIII. No. 12

A MONTHLY JOURNAL for Engineers, Architects, Surveyors and Contractors and all interested in Cement, Concrete, Reinforced Concrete, Fire-Resisting Construction and Structural Steel.

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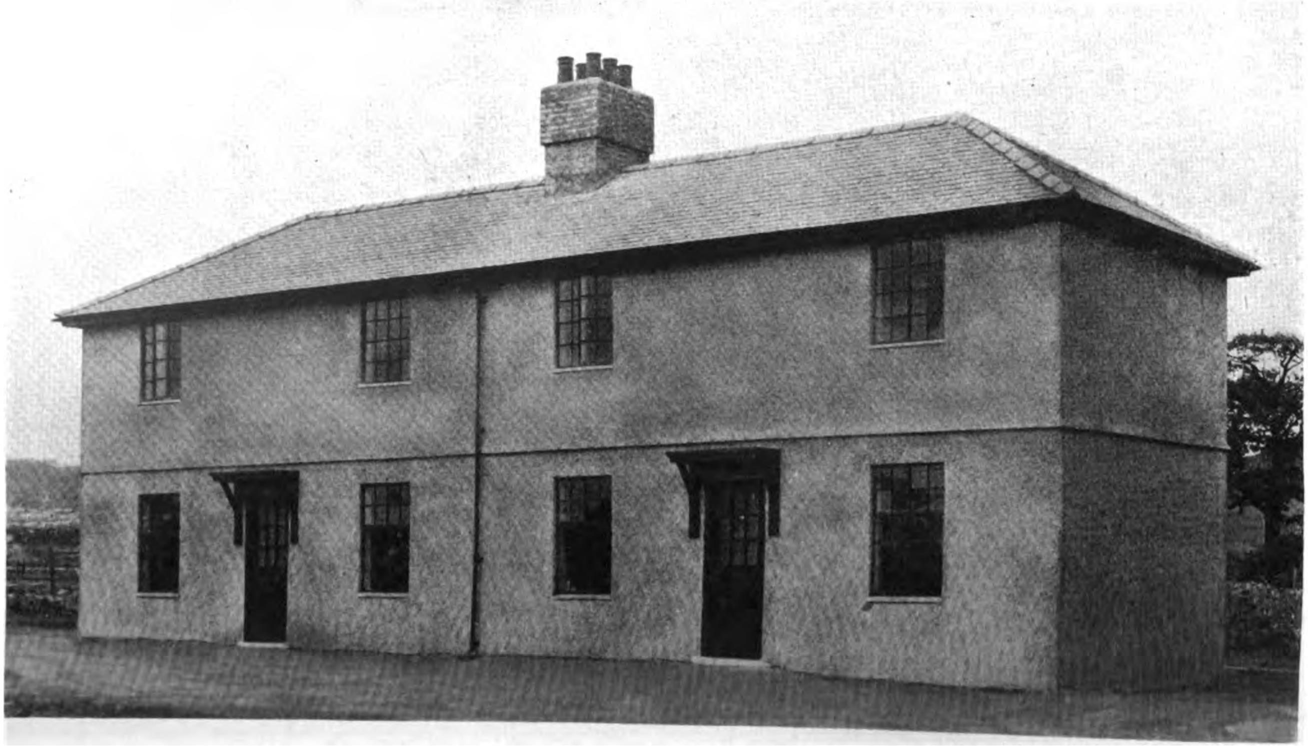

[Mr. H. S. Chorley, Architect.

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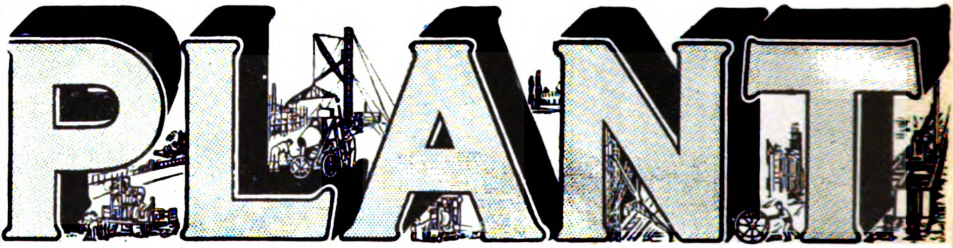

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- Tar and Bituminous Macadam and Asphalt Plants.
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- Winches, Self-Tipping Skips, etc.

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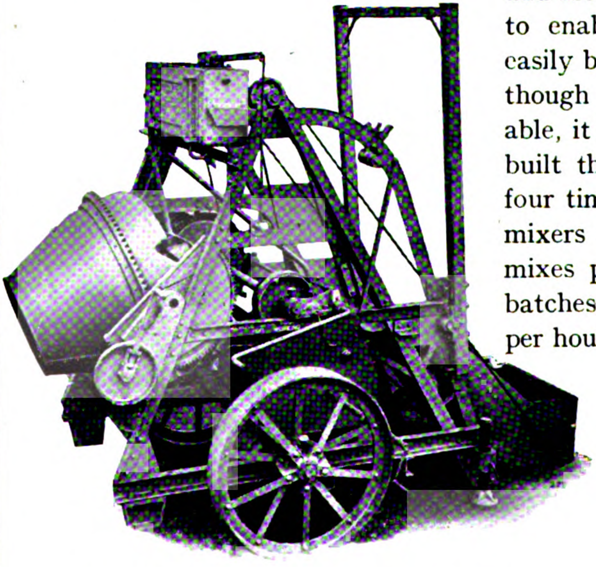
It is self-contained with 7 cu. ft. elevating skip, water tank and high-grade 3 h.p. petrol engine, and mounted on two large wheels to enable it to be pulled about easily by two or three men. Although light and extremely portable, it is nevertheless so strongly built that it lasts two or even four times as long as some other mixers that cost as much. It mixes perfectly and turns out 40 batches of 5 cu. ft. or 7·5 cu. yds. per hour. Can be fitted with winch for hoisting barrows, etc.



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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume XVIII. No. 12. LONDON, DECEMBER, 1923.

EDITORIAL NOTES.

THE QUALITY OF CONCRETE.

A GERMAN contemporary—*Der Bauingenieur*—in reporting some tests on concrete, presents some figures which show that the tensile and compressive strengths of a certain concrete were nearly trebled by rapping the shuttering with pneumatic hammers during the pouring of the concrete, the effect doubtless being a close packing of the concrete and elimination of voids. Without expressing any opinion as to the practical merits of this particular process, it is suggested that any system that will lead to improvement in the quality of concrete is worth careful investigation and encouragement, because of the developments in the application of concrete that may become possible with marked improvement in quality of the material.

It is hardly to be disputed that there is room for improvement in this direction, and inspection of a great deal of concrete work in progress provides evidence of the fact. The detrimental effect of excess of water in a concrete mixture has been proved, and yet there are few jobs to be seen where the proportion of water could not be reduced with advantage to the concrete. Clean and well-graded aggregates are also recognised as essential in high-class concrete, but still there is much concrete obviously weakened by the presence of unsuitable aggregate.

It may be contended that means of improvement tend to add to the cost, that the minimum of water for mixing demands more labour, and that clean aggregates are frequently more expensive than dirty or otherwise unsuitable aggregates, but it is a question for consideration whether the use of the minimum of water and clean aggregate would not in many cases permit of a concrete as good or better being produced with a smaller proportion of cement, and so without a higher cost than with cheap aggregate and a slushy mixture. It is, in fact, not a foregone conclusion that good concrete costs more than indifferent concrete, but if such were the case there still remain the advantages to be gained of improving the quality of concrete for the development of its applications. Concrete roads may be cited as the most prominent example of the development of the concrete industry due to improvement of quality, and it is probably not too much to assert that the concrete roads of to-day have only become possible because of the improvement in the quality of concrete brought about by better methods of concrete production and better quality cement. There are no doubt many factors which will tend to develop the concrete age, but one of these is undoubtedly quality, and it is within the scope of every concreter to aid in this direction by availing himself of the theoretical knowledge that has been obtained and by systematic observation to take every opportunity of producing the best product.

THE NEW HOUSING PROGRAMME.

ALL who are concerned with the housing question, whether as social reformers or as the man in the street wanting a house, will welcome the recent statements in the House of Commons that the new housing programme is making satisfactory progress and that the scheme is being readily adopted by the local authorities. Perhaps because it is farther removed from controversial politics, housing is now proceeding with less publicity than the " Addison " scheme, and, perhaps for the same reason, there seems every probability that a larger number of houses will be erected. Costs have fallen all round, and the £1,000 cottage of three years ago is now being built in the neighbourhood of £400, but there is still the same need for the strictest economy that existed when the first scheme was in progress, and the need for houses has not diminished. It is gratifying, therefore, to know that the responsible authorities are still considering different methods and materials of construction, and that these are being adopted where they are warranted by circumstances. The claims of the concrete cottage are now generally recognised, and the fact that Belfast, Edinburgh, Swansea, and other large centres have decided to use this material in one or other of its different forms is a very practical answer to those who have decried concrete as a material for cottage construction on æsthetic, structural, or other grounds. It has to be recognised that a predilection exists in favour of materials which have been used for centuries, but in face of the acute shortage of dwellings prejudice must not stand in the way of any method which will enable sound and pleasant-looking cottages to be erected in the shortest space of time. Directly after the Armistice the great need for houses brought into the field a vast number of new systems of concrete construction, many of which were inherently uneconomic or unsound propositions, and brought discredit to concrete as a material for building small houses ; but it can safely be said that these have by now been eliminated by the test of time, and that excellent cottages can be built with practically any of the methods which have survived. In this issue we give a description and illustrations of a system which is now being adopted by several large local authorities. As will be seen from the photographs of houses which have been standing for some years, this method lends itself admirably to architectural treatment, and, what is more important, cottages built on this or any other concrete system can be built without the delay which is now inseparable from any method which requires the services of bricks or the bricklayer. Moreover, instead of calling for the services of a trade which has no workers unoccupied, the unemployed can be trained to build concrete cottages in a very short space of time, and thus alleviate at the same time the two most pressing problems with which local authorities are faced—housing and unemployment.

LAYING CONCRETE ROADS.

THE paper read before the Public Works, Roads and Transport Congress by Mr. W. P. Robinson, A.M.Inst.C.E. (County Surveyor of Surrey), and given in abstract in this issue, is of considerable interest in indicating the popularity of the concrete road among road authorities, but it is especially to be welcomed on account of the emphasis it placed on the necessity for roads of this type being carried out to a carefully-prepared specification and under the direction of expert supervision. A sufficient number of concrete roads have now been laid in this country, and have been in use for a sufficient number of years, for their design to have become practically standardised as regards thickness and proportioning and nature of aggregates to secure the best and most economical results for different requirements, but unfortunately this knowledge is still sometimes nullified by the manner in which the work is executed. Although wide publicity has been

THE PACIFIC HIGHWAY: VIEW NEAR BLAINE.
A BRITISH COLUMBIAN CONCRETE ROAD.

given to the results of recent investigations, which have beyond doubt established the fact that an excess of water not only delays the setting and reduces the strength of concrete but is also extremely wasteful of cement, one still finds many roads laid with concrete which is much too wet. For a full discussion on the injurious effects of too much mixing water we would refer to the report by Professor Abrams in our April and May, 1923, issues. The problem is, of course, solely one of supervision, and although there are many capable foremen who are producing excellent results the facts established by the most recent investigations are perhaps not generally known. The factor which is generally responsible for the operators' liking for a " sloppy " mixture, namely, the greater ease of handling, does not apply with such force on road works where mechanical mixers, and in some cases pavers too, are used, and it should not be impossible to instil into the minds of concreters on road work that there is a correct proportion of water to use. We would suggest that the specification should not merely give the proportions of cement, sand, and coarse aggregate to be used, but that the fourth constituent should be taken into account as well; the road would in many cases certainly be better and cheaper if the specification also laid down definitely the consistency of the mixture, either by use of the slump test or by definitely stating that " sloppy " concrete is not to be used.

In the course of his paper Mr. Robinson said a main road recently constructed with concrete throughout had " not yet shown any cracks, nor have the joints been chipped or affected in any way, although the road has been used almost daily for six months by a 14-ton traction engine hauling three trucks of gravel to the work. The surface is absolutely free from corrugation." Later he said, " I am satisfied that an 8-in. layer of reinforced concrete, if properly laid and cured, will give greater satisfaction to the users of the road and be more economical in the end than a tarmacadam or bituminous carpet on a foundation of hardcore or pitched stone." Which shows that careful attention to the execution of the work is amply repaid by results.

FIREPROOF FLOORS

THE matter raised by a correspondent in this issue as to the desirability of standardisation in requirements by the Fire Office Regulations is one that requires consideration. The difficulty arises from the fact that when ordinary reinforced concrete floors are employed the Fire Office Regulations specify that such floors must be designed in accordance with the London County Council Regulations for Reinforced Concrete, but when one or other of the proprietary hollow floors is employed there is no restriction as to the method of design although such floors are in fact usually a form of reinforced concrete construction. The matter is one which in our opinion concerns the Fire Offices and the insurance companies more than the London County Council since the regulations of the latter body apply only to reinforced concrete frame buildings, and have no statutory force for other classes of buildings.

The objection would probably be met if some purely professional body issued a standard specification for reinforced concrete floors, whether monolithic or of the hollow type. We have no doubt that the Fire Offices and the insurance companies would adopt such a standard specification, and all forms of construction would then stand upon an equal footing as to calculations involved in design.

SIMPLE EXAMPLES OF REINFORCED CONCRETE DESIGN.

By **OSCAR FABER, O.B.E., D.Sc.**

(*Continued from April,* 1922.)

[*The author regrets the unavoidable interval which has occurred since the last instalment, and intends to complete the series without a further break if possible. The present number is a continuation of an example of a design of a warehouse of two floors, which, containing many problems of great importance, is being treated at some length.*]

In our last instalment we showed that standard columns 16 in. by 16 in. for the upper length and 24 in. by 24 in. for the lower lengths were stressed to about 600 lbs./in.², whether we considered them fully loaded from all sides or whether we considered them loaded on one side only, when we found a smaller load, but a stress due to bending owing to eccentricity of loading. We concluded that as a rule bending need not be calculated for interior columns.

This is very different in the case of external columns. External columns get their worst load and bending due to eccentric loading simultaneously, and it therefore becomes essential to consider the stresses due to such bending.

COLUMN TYPE (*b*).

We will calculate column type (*b*), being wall columns carrying one main beam and two secondary beams.

The load on wall beams 1 and 2 is as follows:—

Floor level 4 ft. × 25 ft. × 300 lbs. = 30,000 lbs.
Wall load 3 ft. × 1 ft. × 150 × 25 ft. = 11,250 „
41,250 „

The load on main beam 3 is

$$25 \text{ ft.} \times 25 \text{ ft.} \times \tfrac{2}{3} \times 300 = 125{,}000 \text{ lbs.}$$

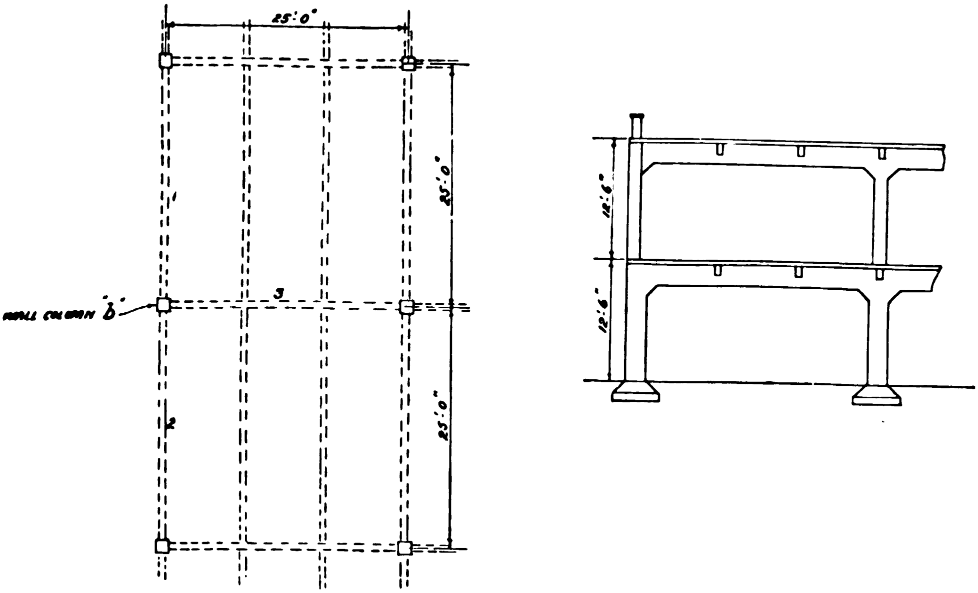


FIG. 11.
SHOWING WALL COLUMN 'b'.

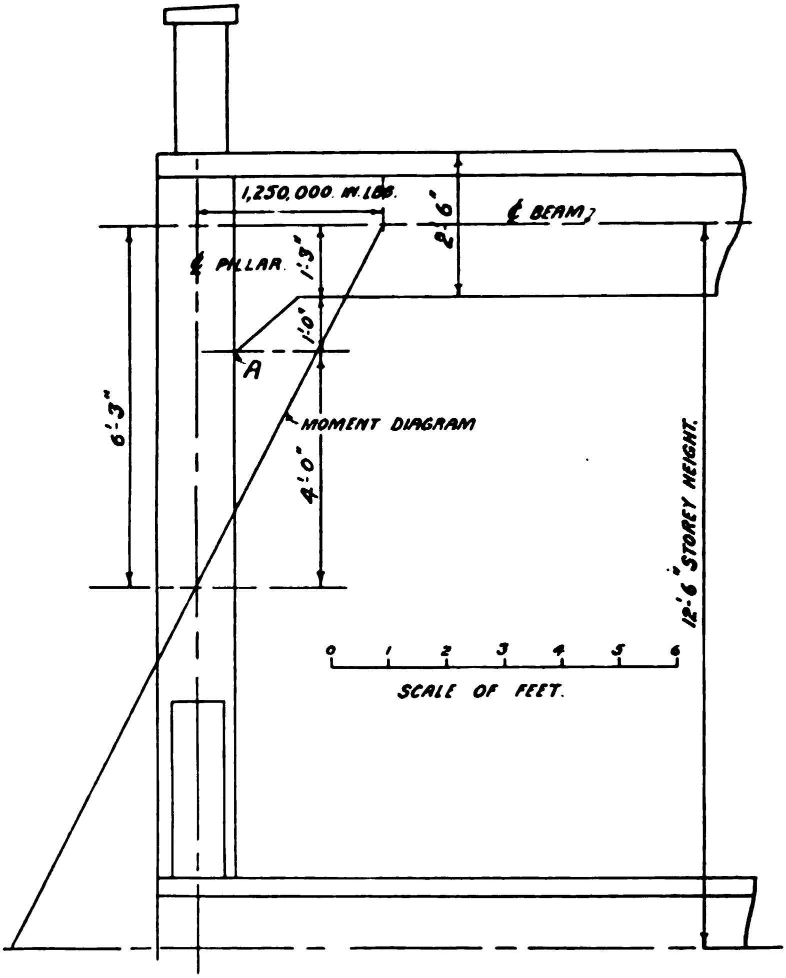


FIG. 12.
MOMENT DIAGRAM IN WALL COLUMN
UPPER TIER.

The reaction on the column is at each floor therefore 20,625 lbs. from each secondary beam, and 62,500 lbs. from the main beam.

The reactions from the secondary beams are, however, balanced and therefore produce no bending of the column, whereas that of the main beam clearly does produce bending.

In general the author recommends making wall columns the same size as interior columns, although they carry less load but more bending, until the fourth story from the top is reached.

Let us, in the present case, adopt standard 16-in. square columns for the upper tier and standard 24-in. square columns for the lower tier (see *R.C.S.E.*, p. 53, for standard columns), as we found required for the interior columns.

Then the direct stress in the upper tier is

$$\begin{array}{llll} 20{,}625 & \text{lbs. from} & \text{beam} & 1. \\ 20{,}625 & \quad ,, & \quad ,, & 2. \\ 62{,}500 & \quad ,, & \quad ,, & 3. \\ \hline 103{,}750 & \text{lbs.} & & \end{array}$$

$$c = \frac{103{,}750}{300} = 345 \text{ lbs./in.}^2.$$

300 being the equivalent area of the 16-in. square column (see *R.C.S.E.*, p. 53).

Similarly the direct stress in the lower tier is

$$c = \frac{207{,}500}{664} = 310 \text{ lbs./in.}^2$$

664 being the equivalent area of the 24-in. column.

UPPER TIER.—Following the calculation for stress due to bending [given in *R.C.S.E.*, Chap. V.] we have

$$F_p \text{ (outside pillars)} = 6$$

$$S_p = \frac{I}{l} \text{ of pillars}$$

$$= \frac{7{,}090}{150} = 47.$$

[7,090 being I for a 16-in. pillar (see *R.C.S.E.*, p. 53), 150 being the length of tier 12 ft. 6 in.]

$$S_b = \frac{I}{l} \text{ of beam}$$

$$\frac{78{,}000}{300} = 260$$

Whence

$$\frac{F_p.S_p}{S_b} = \frac{6 \times 47}{260} = 1{\cdot}09.$$

Whence bending moment in column is

$$\cdot 025 Wl \times \frac{12}{9}$$

$$= \cdot 025 \times 125{,}000 \times 300 \times \frac{12}{9}$$

$$= 1{,}250{,}000 \text{ in. lbs.}$$

[NOTE.—In the above, ·025 is obtained from Table II, *R.C.S.E.*, p. 54, W is the total load on the beam producing bending in the column, l its span in inches, while $\frac{12}{9}$ is a coefficient required for third point loading.]

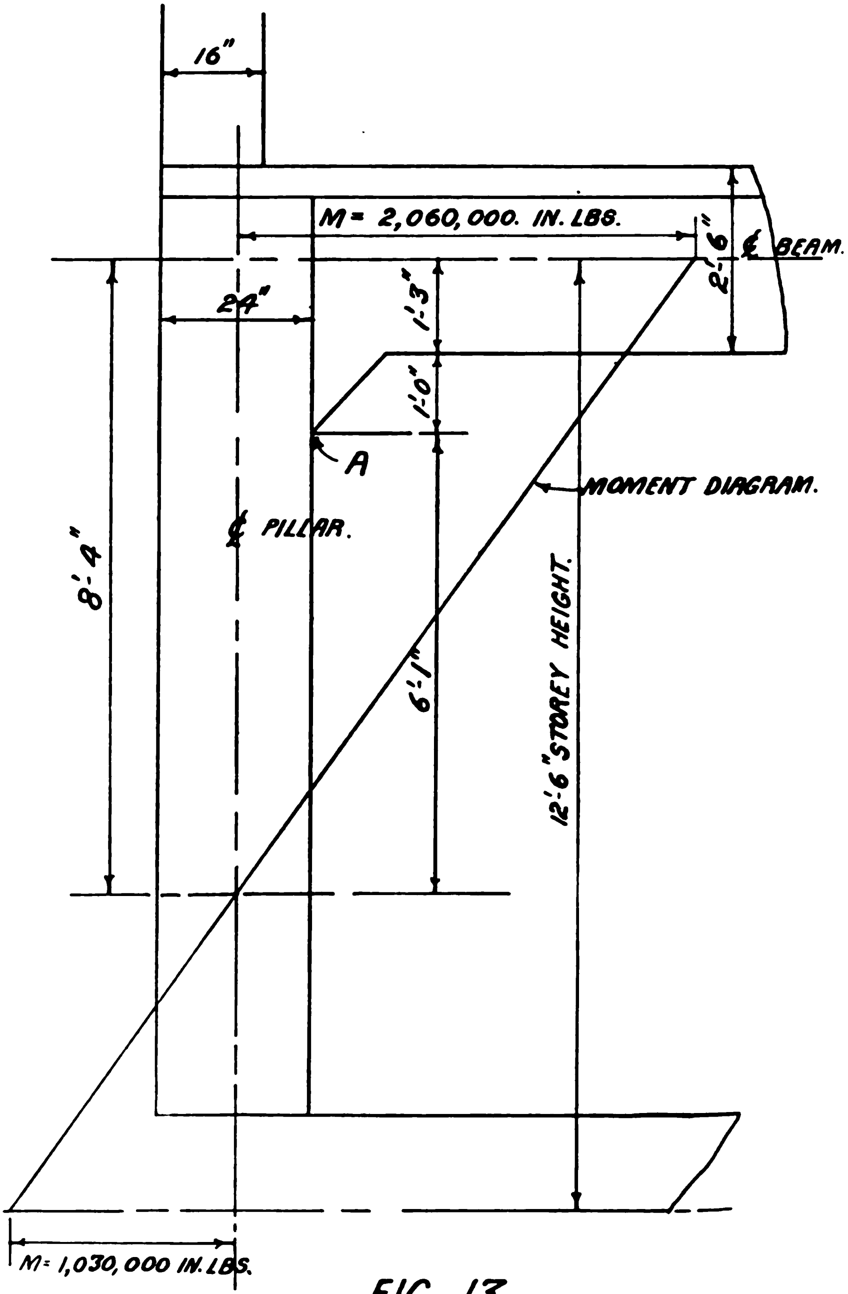


FIG. 13.

MOMENT DIAGRAM IN WALL COLUMN LOWER TIER

The stress due to bending is

$$c = \frac{Bd}{2I} \text{ (see } \textit{R.C.S.E.}\text{, p. 55)}$$

$$= \frac{1{,}250{,}000 \times 16 \text{ in.}}{2 \times 7{,}090} = 1{,}400 \text{ lbs./in.}^2$$

This stress may be reduced for the reduction in moment which takes place between the centre line of beam and the point A (see *Fig.* 12), which is clearly the point of worst stress, since above this the section increases rapidly.

With this correction, we may take the stress due to bending as

$$c = 1{,}400 \times \frac{4}{6\frac{1}{4}} = 900 \text{ lbs./in.}^2$$

and the maximum stress as

$$c = 345 \text{ direct} + 900 \text{ bending}$$
$$= 1{,}245 \text{ lbs./in.}^2$$

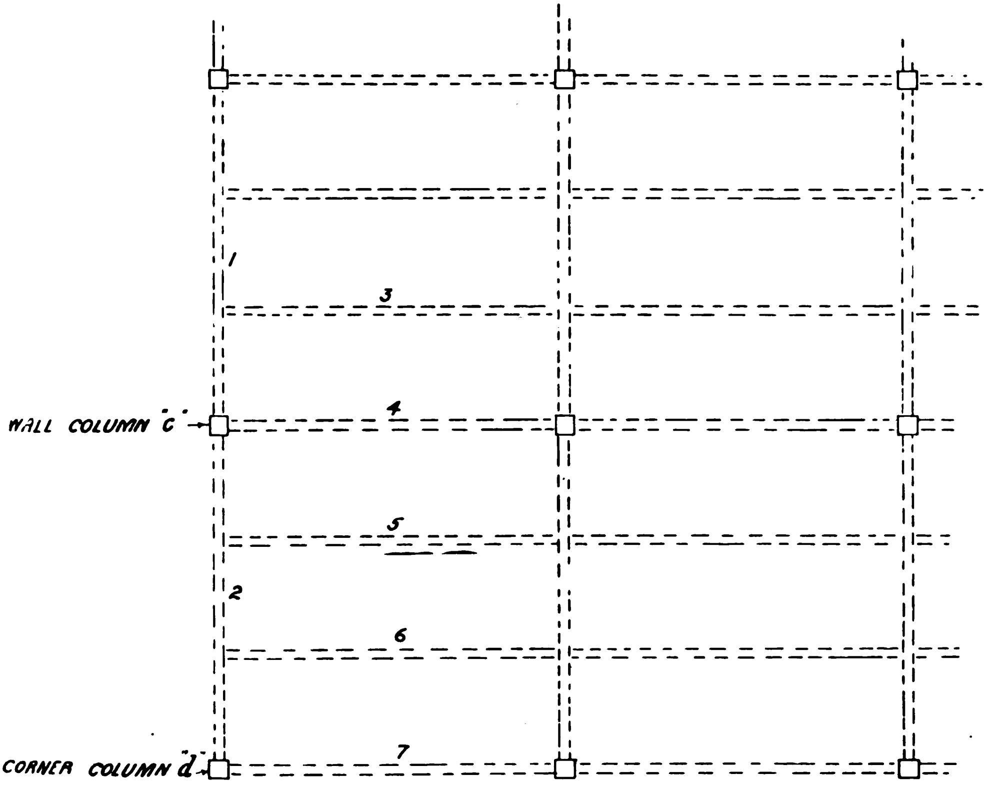


FIG. 14
SHOWING WALL COLUMN c
AND CORNER COLUMN "d"

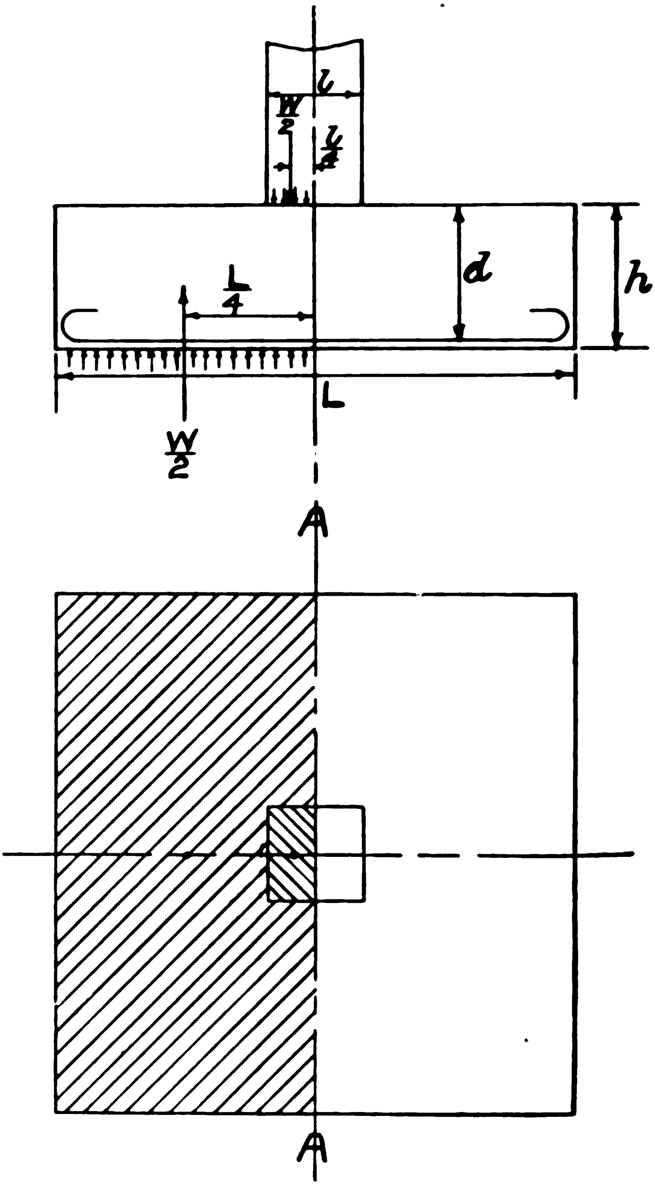


FIG 15.

The author would not consider such a stress dangerous, provided a 1 : 1 : 2 concrete were used, since when bending is taken into account it is clearly absurd to ask for the same factor of safety as when it was ignored.

LOWER TIER.—The only unbalanced load is the main beam reaction of 62,500 lbs. as before. The bending in the column due to it is resisted by both the upper and lower tiers of column, as follows :—

$$F_pS_p \text{ (upper length)} = 6 \times \frac{7{,}090}{150} = 282$$

$$F_pS_p \text{ (lower length)} = 4 \times \frac{34{,}000}{150} = 904$$

$$F_pS_p \text{ total} = 1{,}186$$

The value of F_p for the upper length is taken as 6 because the top beam is deflected in the same way as the lower beam, while F_p for the lower length is taken as 4 because the footing is assumed horizontal, or undeflected by the bending of the beams.

$$S_b \text{ as before is } \frac{7{,}800}{300} = 260$$

whence

$$\frac{F_p \,.\, S_p}{S_b} = \frac{1{,}186}{260} = 4 \cdot 5.$$

$$\text{Total moment in column} = \cdot 053 \times 125{,}000 \times 300 \times \frac{12}{9} = 2{,}650{,}000 \text{ in. lbs.}$$

[Note ·053 is from Table II, *R.C.S.E.*, p. 54.
W = total load on beam bending the columns.
l = span in inches of this beam.
$\frac{12}{9}$ = factor to allow of third-point loading as before.]

Of this $2{,}650{,}000 \times \frac{904}{1{,}186} = 2{,}060{,}000$ in. lbs. is carried on the lower tier column just under the lower floor, producing a stress of

$$c = \frac{Bd}{2I} \text{ (see } R.C.S.E., \text{ p. 55)}$$

$$= \frac{2{,}060{,}000 \times 24 \text{ in.}}{2 \times 34{,}000} = 720 \text{ lbs./in.}^2$$

This can be reduced for depth of beam and haunch as before, see *Fig.* 13.

$$\text{Stress at } A = 720 \times \frac{8 \text{ ft. } 4 \text{ in.} - 2 \text{ ft. } 3 \text{ in.}}{8 \text{ ft. } 4 \text{ in.}}$$

$$= 720 \times \frac{73}{100} = 520 \text{ lbs./in.}^2$$

whence max. stress

c = 300 lbs./in.^{2} direct + 520 lbs./in.^{2} due to bending.
= 820 lbs./in.^{2} total.

NOTE.—The moment diagram in *Fig.* 13, in which the moment at the bottom is half that at the beam level, assumes that the foundation remains fixed. (See *R.C.D.*, Vol. 1, p. 70.)

COLUMN TYPE (*c*).

Columns Type (*c*) are wall columns carrying two main beams and one secondary beam. The calculation for this is on exactly similar lines to Type (*b*), and therefore no useful purpose will be served here by making them.

The total load is practically the same, while the bending is less, since the two main beams 1 and 2 are balanced about the column and only the secondary beam 4 produces bending. Hence the stresses will be less than for columns (*b*) if the same sections are retained. In the calculations for bending, the factor $\frac{12}{9}$ in the last case will not occur as the load on the secondary beam is a distributed one, and not two point loading. An interesting point is whether beams 3 and 5, which clearly also deflect, produce bending on the column. Clearly if main beams 1 and 2 had great torsional strength and torsional stiffness or rigidity, such bending would be transmitted to the column, but in most cases the torsional stiffness of the main beam is so small compared with the bending stiffness that the effect of the bending of beams 3 and 5 is quite negligible on the column.

COLUMN TYPE (*d*).

Type (*d*) are the four corner columns in the building. Each of these carries one external main beam (2), and one external secondary beam (7). The loads on these (from previous calculations) are 73,700 lbs. and 41,250 lbs. respectively, making the column load at each floor half these figures:

$$36{,}850 + 20{,}625 = 57{,}475 \text{ lbs.}$$

This example is worth working out, because this column is subject to bending in two planes at right angles by the deflection of beams 2 and 7 respectively.

The direct stresses are

$$\frac{57{,}475}{300} = 192 \text{ lbs./in.}^2 \text{ in the upper tier.}$$

[57,475 being the load in lbs. and 300 the equivalent area of the standard 16 in. column]

and $$\frac{114{,}950}{664} = 173 \text{ lbs./in.}^2 \text{ in the lower tier.}$$

[114,950 being the load in lbs., and 664 the equivalent area of the standard 24 in. column.]

Upper Tier.—After what has been done, the calculations need less explanation. But the wall beams, being rectangular beams above the slab forming a parapet on the upper floor and a wall under the window on the lower floor, are not standard T-beams, and therefore their moment of inertia I cannot be taken from the Table on page 35 (*R.C.S.E.*) but need to be calculated.

The main wall beam was described in the April, 1922, issue as being 36 in. by 12 in. above a 5 in. slab ($h = 41$ in.) and having six 1 in. rods top and bottom and a resisting moment of 2,360,000 in. lbs.

The most convenient method of calculating the moment of inertia in such a case is

$$I = \frac{Ry}{f}$$

where R is the resisting moment,

y is the distance of any point from the neutral axis,

f is the concrete stress at that point or the steel stress divided by 15.

Applying this in the present case and considering the top fibre, this is at a distance of $n = \cdot 36d = \cdot 36 \times 38 = 13\cdot 7$ in. from the neutral axis.

Whence $I = \frac{2{,}360{,}000 \times 13\cdot 7}{600} = 54{,}000$ in. units.

Alternatively, calculating it from the steel in tension, its distance from the neutral axis is

$$d - n = 38 \text{ in.} - 13\cdot 7 \text{ in.} = 24\cdot 3 \text{ in.}$$

$$\text{whence } I = \frac{2{,}360{,}000 \times 24\cdot 3 \times 15}{16{,}000}$$

$$= 54{,}000 \text{ in. units.}$$

The secondary wall beam would be the same size, for general convenience, but less steel would be sufficient, and we will assume for our calculations that it has a moment of inertia of

$$I = 27{,}000 \text{ in. units.}$$

Bending due to Main Wall Beam.—

$$S_p = \frac{I}{l} \text{ of pillar (upper tier 16 in. standard pillar)}$$

$$= \frac{7{,}090}{150} = 47 \text{ as before.}$$

$$S_b = \frac{54{,}000}{300} = 180$$

$$\left[\begin{array}{l}\text{54,000 being } I \text{ of wall beam.}\\ \text{300 being its span in inches.}\end{array}\right]$$

F_p for outside pillars being 6.

whence

$$\frac{F_p.S_p}{S_b} = \frac{6 \times 47}{180} = 1{\cdot}57$$

whence bending in column

$$B = {\cdot}03 \times 73{,}700 \times 300 \times \frac{12}{9}$$

$$= 880{,}000 \text{ in. lbs.}$$

[·03 is from Table II, *R.C.S.E.*, p. 54.]

Stress in column due to bending

$$c = \frac{Bd}{2I}$$

$$= \frac{880{,}000 = 16 \text{ in.}}{2 \times 7{,}090} = 1{,}000 \text{ lbs./in.}^2$$

This can be reduced for depth of beam as before (though this time there is no haunch)

$$c = 1{,}000 \times \frac{6 \text{ ft. } 3 \text{ in. } - 1 \text{ ft. } 8 \text{ in.}}{6 \text{ ft. } 3 \text{ in.}}$$

$$= 1{,}000 \times \frac{53}{73} = 725 \text{ lbs./in.}^2$$

Bending due to Secondary Wall Beam.—

$$S_p = 47 \text{ as before}$$

$$S_b = \frac{27{,}000}{300} = 90$$

$$F_p = 6$$

$$\frac{F_p.S_p}{S_b} = \frac{6 \times 47}{90} = 3{\cdot}14$$

$$B = {\cdot}04 \times 41{,}250 \times 300$$

$$= 500{,}000 \text{ in. lbs.}$$

stress in column due to bending

$$c = \frac{Bd}{2I} = \frac{500{,}000 \times 16 \text{ in.}}{2 \times 7{,}090} = 564.$$

This can be reduced as before for depth of beam to

$$c = 564 \times \frac{53}{73} = 410 \text{ lbs./in.}^2$$

Hence total maximum stress in column is

c = 192 lbs./in.^{2} direct.
+ 725 lbs./in.^{2} bending from main wall beam.
+ 564 lbs./in.^{2} bending from secondary wall beam.

1,481 lbs./in.^{2}

The student will now have no difficulty in calculating the stress in the lower tier, which will be less than the above.

The instructive thing which emerges from these calculations is that though all the columns are made the same size the stresses are least on the interior ones ($c = 688$), greater on wall columns Type (*b*) ($c = 1{,}245$) and greatest on corner columns Type (*d*) ($c = 1{,}481$) although the actual loads are least on the corner columns, greater on the wall columns, and greatest on the interior ones.

[NOTE.—The above stresses are for the upper tiers.]

FOOTINGS.

If a square footing of length L carries a column of length of side l, the projecting parts of the footing will be subject to a bending moment which is greatest on the central section such as AA, see *Fig.* 15.

If we consider the total upward load on the left-hand side of this section, it is clearly $\frac{W}{2}$—where W is the total load on the footing—and similarly the total downward load on the left-hand side (occurring in the left-hand side of the column) is also $\frac{W}{2}$. If we assume constant stress distribution on the column and on the foundation soil, then the distances from the section AA (or centre line) of these upward and downward resultants are clearly $\frac{L}{4}$ and $\frac{l}{4}$ respectively as shown in *Fig.* 15, whence the moment on AA is

$$B = \frac{W}{2} \times \frac{L}{4} - \frac{W}{2} \times \frac{l}{4}$$

$$\text{or } B = \frac{W}{8}(L - l)$$

This is a very simple and accurate formula, which enables us to calculate such footings with great ease.

EXAMPLES.

The maximum load on interior columns was 375,000 lbs.

Assuming that the soil will safely carry 2 tons per sq. ft., the area of footing required would be

$$A = \frac{375{,}000}{2 \times 2{,}240} = 83{\cdot}2 \text{ sq. ft.}$$

$$\text{whence } L = \sqrt{83{\cdot}2} = 9 \text{ ft. approximately.}$$

From previous calculations,

$$l = 24 \text{ in.}$$

$$\therefore B = \frac{W}{8}(L - l) = \frac{375{,}000}{8}(108 \text{ in.} - 24 \text{ in.})$$

$$= 3{,}950{,}000 \text{ in. lbs.}$$

If we resist this by a rectangular slab 108 in. wide, reinforced with ·675 per cent. of steel, then the resistance moment will be—

$$R = 95\, bd^2$$

$$= 3{,}950{,}000 \text{ in. lbs.}$$

$$\text{whence } d = \sqrt{\frac{3{,}950{,}000}{95 \times 108}}$$

$$= 19{\cdot}6$$

$$\text{and } h = 22 \text{ in. to allow reasonable cover.}$$

The area of steel required is given by—

$$A = \frac{\cdot 675}{100}\, b.d$$

$$= \frac{\cdot 675}{100} \times 108 \times 22$$

$$= 16 \text{ ins.}^2$$

16 $1\frac{1}{8}$ in. diameter rods give the requisite area.

Now, clearly the centre section at right angles to *AA* would be subject to exactly the same moment, and therefore an additional 16 $1\frac{1}{8}$ in. rods would be required in the other direction.

It will be unnecessary here to calculate the footings for the other columns, as they follow exactly the above example.

A few questions are worth discussing shortly, as they raise points which an intelligent designer may raise.

First, as the columns may be subject to moment at the bottom under certain conditions, should not the footing also be so designed? The answer is, strictly speaking, yes; but an eccentricity which produces a large effect on a 24-in. column produces only a very small effect on a 9-ft. footing and may be neglected in most cases.

Second, is the footing illustrated the most economical type? The author finds it is for footings up to about 10 ft. square.

Some designers use a footing of pyramidical shape. The disadvantage of this is twofold. The sloping upper surface is either to be formed by centering, in which great additional cost is incurred which easily balances the cost of the concrete saved. Also this top centering tends to lift when the concrete is placed and has to be weighted down and is generally troublesome.

Sometimes it is attempted to construct the sloping concrete without centering by mixing it very dry. This frequently results in very poor honeycombed concrete and is a dangerous expedient.

The pyramidical form also requires to be much deeper, because the central section, instead of having the full width available for compression, has the two sides cut away on top just where the compression stress would occur, and a much narrower width has to be reckoned on.

(*To be continued.*)

CONCRETE TUBES FOR INTERNAL PRESSURE.

By Dr. F. EMPERGER.

(*Continued from November issue.*)

TESTS ON SMALL TUBES.

THE tubes with which the experiments were carried out had an internal diameter of 12 in., and the reinforcement consisted of a wire of 2 mm. diameter ($\frac{1}{12}$ in.) wound round the tube at a pitch of 1 cm. (about $\frac{3}{8}$ in.) at a uniform tension, this making the reinforcement almost exactly 1 per cent. The material used was an iron wire and a steel wire of an ultimate tensile strength of 45 tons and 82 tons per sq. in. respectively. The method employed for winding the wire under uniform tension was very simple, the tube being placed so as to allow it to rotate horizontally, while the wire was wound on a drum rotating parallel with the tube. The ends of the tube were closed with steel plates, to one of which the end of the wire was fastened, and then wound on the tube by rotating the latter, while the drum was subject to a brake pressure the magnitude of which determined the tension in the wire. When the tube was fully wound, the other end of the wire was fixed to the steel plate at the other end. The maximum initial stress in the wire, according to equation (23), is

$$S_0 = \frac{100{,}000 - 15{,}750}{1 \cdot 6} = 52{,}500 \text{ lbs. per sq. in.}$$

which is equivalent to a tension of 260 lbs. in the wire during winding. The corresponding figures for the steel wire are

$$S_0 = 106{,}000 \text{ lbs. per sq. in.}$$

while the tension in the wire is 520 lbs.

The actual wire tension employed during the tests amounted to 112 lbs. and 224 lbs. for each of the two iron wires, and 560 lbs. for the steel wire, the first two tests being somewhat below the figure arrived at by the above calculation, the third slightly above the value required for steel. This initial tension resulted in a tensile stress of 23,000, 46,000, and 115,000 lbs. per sq. in. respectively in the wire, and a corresponding compression stress in the concrete tube of 240, 490 and 1,200 lbs. per sq. in.

On the basis of these initial stresses it is possible to calculate the concrete stress under the working pressure of the tube, assuming a factor of safety of 1·5 for the latter, and a 2½-times safety for the tensile stress in the concrete, as provided in equations (15–18). The concrete stress is found as 200, 170, and 28 lbs. per sq. in. respectively, from which it will be seen that in the second case the initial stress is nearly sufficient to secure the 2½-times factor of safety (140 lbs. as compared to c_u = 350 lbs. per sq. in.), while in the third case the initial stress is higher than necessary; an initial tension in the wire of about 270 lbs. would have given the working stress required. It is possible to neglect the tensile strength of the concrete altogether by increasing the initial stress; assuming an ultimate concrete strength of 285 lbs. per sq. in. this result would be obtained with an initial concrete stress of 1,400 lbs. per sq. in. and a wire tension of 670 lbs.

The latter case is not really of interest except where cracks have appeared in the tubes of a pressure main or a reservoir; the difficulty in removing the cracked tube and replacing it by a new one may be overcome by using this method, which does not necessitate the removal of the tube or interference with working conditions. In order to obtain this result in the case of the tubes under test it would have been necessary to employ a stronger wire or material of still higher tensile strength. This condition may, however, be said to have existed in the one test carried out with a wire tension of 520 lbs., as the value of m_0 is much greater than 60, the figure which has been assumed throughout the calculations.

In estimating the value of m_0 the fact must not be lost sight of that the latter is not identical with the ratio between the elastic deformations of concrete and of steel. In the case in question it is required to find the deformation ratio which obtains when the concrete is being unloaded from its initial stress, the permanent deformation playing a very important part. In order to find the value of m_0 experimentally, two tubes were employed which had been tested to cracking; in both tubes the tensile

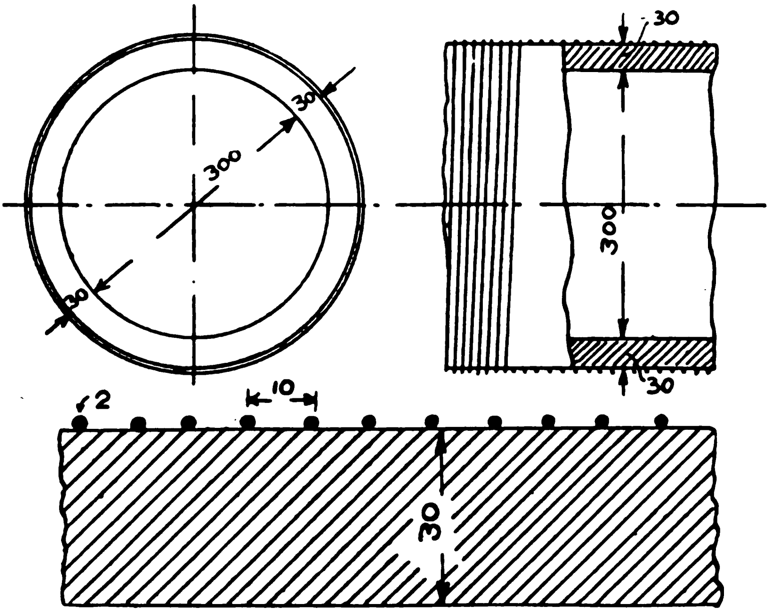


FIG. 2.

strength of the concrete could therefore be considered as non-existent, and the load necessary to neutralise the initial stress may be found from equations (15) and (16) with $c_u = 0$,

$$P = pr = m_o c_o A$$

or, if the value of c_o is introduced from (12),

$$P = pr = m_o k s_o A = m_o k T.$$

In this equation m_o is the only unknown quantity, and

$$m_o = \frac{P}{kT} = \frac{pr}{kT} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (29)$$

T is the total initial tension in the reinforcement per inch length of tube; with wires at 1 cm. pitch and a tension of 112 lbs. and 560 lbs. respectively, T is 282 lbs. and 1,410 lbs. The pressure p at which the cracks started opening again was found by two tests to be 4 atmospheres and 15 atmospheres (57 and 215 lbs. per sq. in.) respectively. From these figures m_o is found as

$$m_o = \frac{57 \times 6}{0{\cdot}01 \times 282} = 120, \text{ and } m_o = \frac{215 \times 6}{0{\cdot}01 \times 1{,}410} = 90.$$

The use of $m_o = 60$ as an average would therefore appear to be quite safe and to allow for any contingencies. Similar considerations prompted the use of the generally accepted $m = 15$, which was proposed twenty-five years ago by the author in the technical journals at that time.

Below is given a comparison between the ultimate strength of the test tubes, as calculated and as found experimentally. The equation (16) gives

$$p'r = c_u t + c_u m A + m_o c_o A$$

$$\text{or } p' = \frac{t}{r} c_u + m \frac{A}{r} c_u + m_o \frac{A}{r} c_o \text{ or, using (13)}$$

$$p' = \frac{t}{r} c_u + m \frac{A}{r} c_u + m_o \frac{A}{r} k s_o$$

$$\text{or } p' = \frac{t}{r} c_u + m \frac{A}{r} c_u + m_o \frac{k}{r} T$$

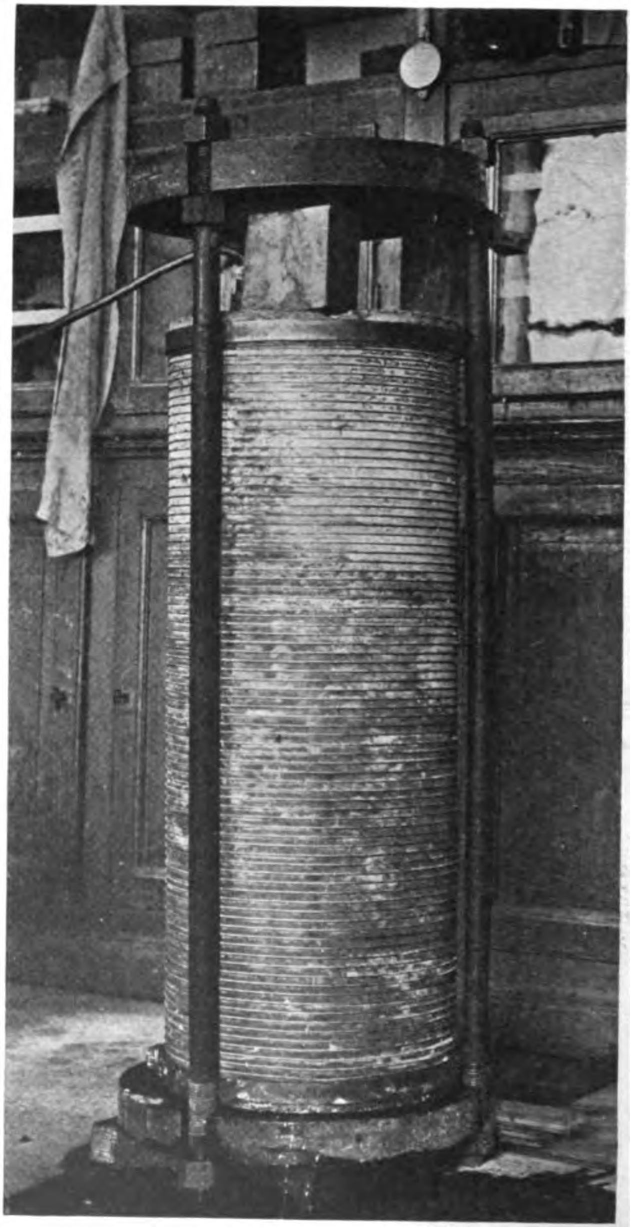


Fig. 3.

where T, as before, is the total initial tension per inch length of tube ; in the three cases T is 282, 564, and 1,410 lbs. respectively. With a c_u of 355 lbs. per sq. in., a thickness of 3 cm. (1·2 in.) and r = 6 in. the equation is reduced to

$$p' = \frac{1{\cdot}2}{6} \times 355 + 45 \times \frac{0{\cdot}012}{6} \times 355 + 60 \times \frac{0{\cdot}01}{6} \times T$$

$$\text{or } p' = 71 + 32 + 0{\cdot}1T = 103 + 0{\cdot}1T\,;$$

for the three cases of T

T = 282 ; p' = 103 + 28·2 = 131 lbs. per sq. in. or 9·3 atm.
T = 564 ; p' = 103 + 56·4 = 159 lbs. per sq. in. or 11·3 atm.
T = 1,410 ; p' = 103 + 141 = 244 lbs. per sq. in. or 17·3 atm.

The tests gave in the three cases 8, 12 and 18 atmospheres respectively, which results show a satisfactory agreement with the calculation.

In order to form a clearer idea of the practical value of the " Emperger-Umlauf " system, a comparison between the latter and ordinary reinforced concrete construction may be useful. Assuming a minimum ultimate strength of the concrete of 285 lbs. per sq. in. and a corresponding allowable stress of 115 lbs. per sq. in., the strength of tubes identical with those tested, but with the reinforcement arranged in the usual way (inside the thickness of the walls instead of outside, and without initial tension) is obtained from equation (2),

$$2{\cdot}5\,P = p'r = c_u(t + mA) = 285(1{\cdot}2 + 45 \times 0{\cdot}012)$$

$$p' = \frac{495}{6} = 82 \text{ lbs. per sq. in.} = 5{\cdot}8 \text{ atm.}$$

It may be noted that in this case it makes no difference whether iron or steel is used. The allowable pressure for the tube is therefore 2·3 atmospheres, which is arrived at on the assumption of c_u = 285 lbs. per sq. in., a high value considering that the concrete cannot be tamped very well between the reinforcement.

A comparison between these figures and those obtained from the plain concrete tubes shows hardly any increase in strength over the latter, mainly because the ultimate strength of the concrete in the tube without reinforcement is greater. If, on the other hand, the figures are compared with the result of the third test, which shows that an ultimate load of 18 atmospheres can be obtained, corresponding to an allowable working load of 12 atmospheres, then it appears that according to this system it is possible to increase the working load 5 times with the same tension stresses in the concrete. That the proposed method of calculating the new system is on the safe side is shown clearly by comparing the calculated figures with the test results ; according to the former the initial compression in the concrete should be neutralised at an internal pressure of 141 lbs. per sq. in. or 10 atmospheres, and therefore it might be expected that cracks would begin to form at this pressure ; the tests showed, however, that this only happened when a pressure of 15 atmospheres was reached. This may be explained by the fact that the modular ratio m_0, which has been taken as 60 in the calculation in order to be on the safe side, is really too low and might well have been increased.

How well the steel is utilised may be seen by calculating the ultimate water pressure that the reinforcement alone might stand, neglecting the presence of the concrete. Assuming the ultimate tensile strength of the steel wire to be 185,000 lbs. per sq. in. while the steel area per inch length of tube is 0·012 sq. in., the ultimate internal pressure taken by this steel would be

$$p' = \frac{185{,}000 \times 0{\cdot}012}{6} = 370 \text{ lbs. per sq. in. or 26 atm.}$$

so that the test result of 18 atmcspheres corresponds to an efficiency of two-thirds of the steel strength. What this means is best seen by trying to visualise a steel tube with the same efficiency—this would correspond to a thickness of about $\frac{1}{83}$ in. of the steel tube, which is obviously unattainable.

In comparing these tubes with steel tubes there are certain points which must not be overlooked. It is, as will appear from the figures above, impossible to keep the

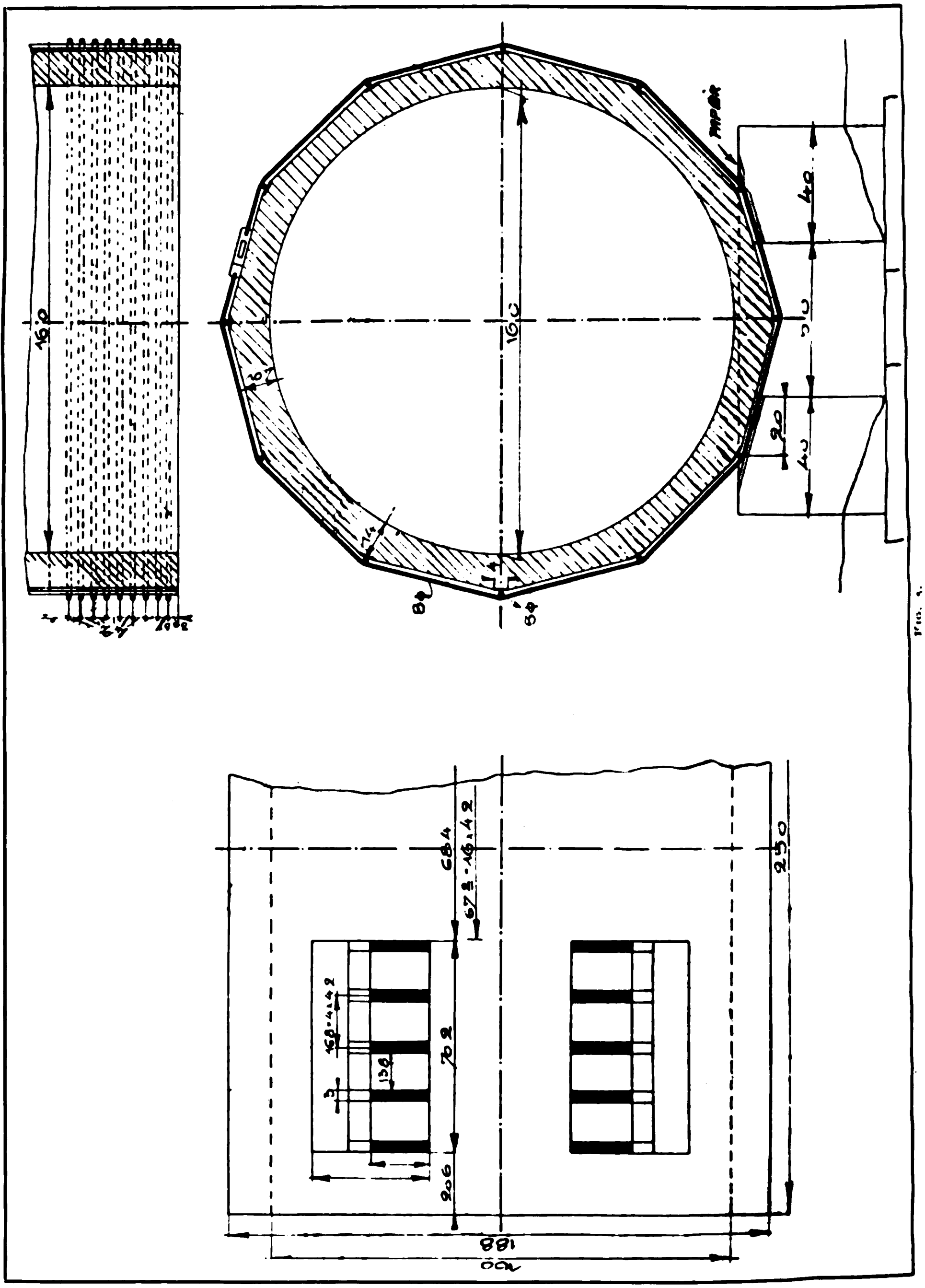

16.0
160
Papier
40
50
40
20
6
14
8ø
8ø
684
678·16,42
702
206
150·4,42
130
3
250
188
200

Fig. 4.

steel tube within the dimensions required by considerations of strength alone, for practical reasons, and a certain allowance in thickness must also be made to counteract the effects of corrosion. Any coating or painting of the tube with a view to diminishing the risk of injury from this source will have to be renewed occasionally, while in the case of a concrete tube the concrete acts as a corrosion preventive. The tests have in any case shown that the new system enables the concrete pressure tube to enter successfully fields which have hitherto been monopolised entirely by the steel tube.

A very important point in the construction of these new pressure tubes is the formation of the joints. However good and efficient, a joint will always constitute extra expense, and it is therefore natural to endeavour to manufacture the tubes in as long lengths as possible, so as to decrease the number of joints. Reference may be made in this connection to the manufacture of tubes in France, described recently in this journal, where, by the use of high-class cement and compressed concrete of a high tensile strength, tubes are easily made 10 ft. long, in spite of the walls being very thin. The usual construction of the joint is shown in *Fig.* 4.

In executing a considerable length of pressure piping on this principle it would appear expedient to diminish the cost of transport by making use of a movable factory in which to make the concrete tubes, fitting them with a steel strip at each end to which to attach the wire, which is then wound with the requisite amount of tension. After being placed, the tubes are joined and the whole of the reinforcement covered by a rendering which serves as protection against rust.

TESTS ON LARGE TUBES.

After having tested the small tubes the question arose as to whether the results gained thereby might be used straightaway on tubes of large diameters, such as are used, for instance, in water-power stations. As several subordinate causes and circumstances might easily influence the results it was considered advisable to prepare test tubes of a diameter equivalent to that of reinforced concrete tubes already in commission, for the purpose of comparing results. As a basis for the dimensions the tubing in use at the Blumau power station was taken, having an internal diameter of 5 ft. 4 in. and being subject to a water pressure occasionally reaching 3 atmospheres. The test tube was prepared at the same time and in the same yard as several other reinforced concrete tubes which were also to be tested. On account of the high cost it was decided to make only one test tube and test it in as many ways as possible so as to eliminate any possible constructional mistakes, and use the experience gained thereby when testing the next complete series, the results of which it is hoped to publish at a later date. That this was a necessary precaution was proved abundantly during the preparations; many weeks of study and alterations were spent before the test tube was gradually improved so as to be in a form that enabled accurate and useful readings to be taken.

The tube in question was 8 ft. 3 in. long, with an internal diameter of 5 ft. 4 in., the external shape being polygonal with 12 sides, the corners of which were protected

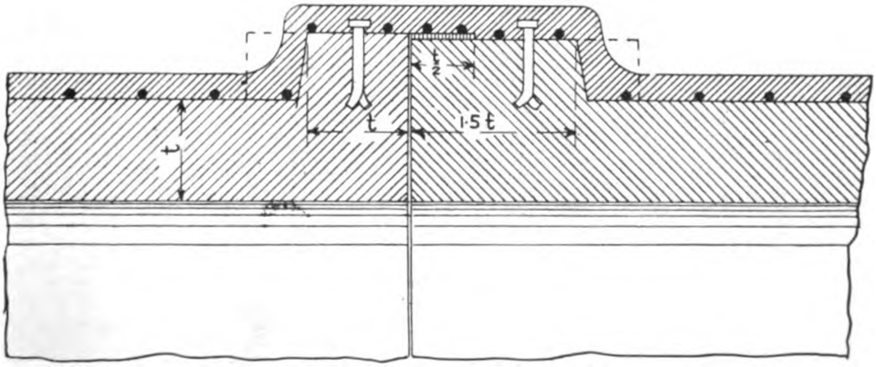


FIG. 4.

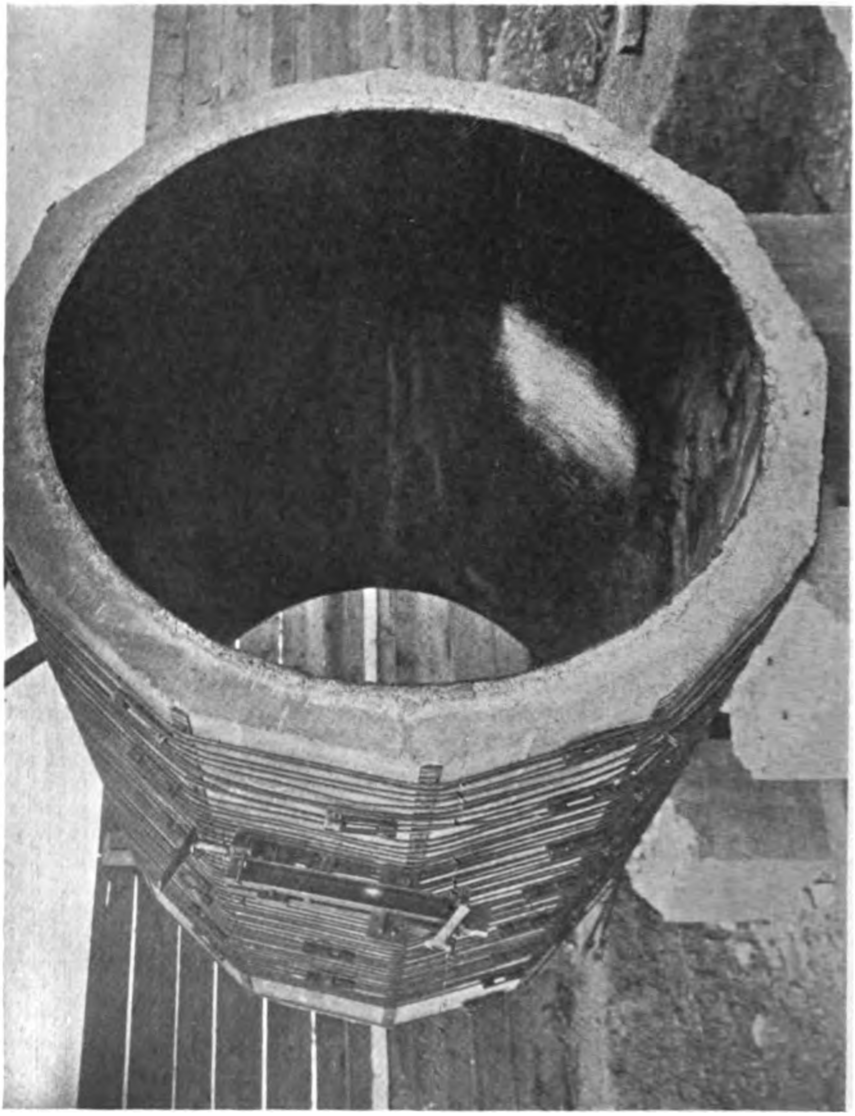


Fig. 6.

by steel flats acting as longitudinal reinforcement. The whole of the testing apparatus was designed at the technical high school in Vienna under the direction of Dr. Rinagl, and the general lay-out is seen in *Fig.* 7. The ends of the tube were closed by means of dished boiler ends, which were suspended from a light trestle so as to avoid any stresses in the concrete due to the weight of the steel plates ; the plates were kept together with eight bolts along the circumference, and one in the centre, and were fitted with the usual manhole. The end plates had an angle iron riveted outside the circumference, and a watertight joint between this angle and the concrete tube was obtained by means of a rubber tube, filled with glass balls to prevent it from slipping under pressure. It was the intention to reinforce the tube immediately after the striking of the centering, in order to show that it was able to take working loads while still green ; on account of the many alterations and trials, however, the concrete was two months old before a real pressure test could be carried out, the arrangement for testing being somewhat elaborate in order to obtain accurate observations, especially as regards repetition of loading and unloading. The inner surface of the concrete tube was rendered and treated with a covering of a special waterproofing material, which proved quite satisfactory. This method of securing watertightness in the concrete is, however, far too expensive and quite unnecessary for the lower pressures dealt with during the tests ; it was only used in order to obtain a fair comparison with the other pressure tubes being manufactured at Blumau, and which used this treatment for the internal concrete surfaces. Before the real test could be undertaken certain preliminary investigations had to be made at the laboratory of the technical high school.

(*a*) DISTRIBUTION OF STEEL STRESS.

In regard to the method adopted of placing the reinforcement in single wires, each kept in tension by a special key, the question arose as to whether it would be sufficient to tighten the wire in one place and whether the tension produced by this process would be sufficiently uniformly distributed to all the other parts of the wire. In order to investigate this point a wheel was constructed on the circumference of which an 8 mm. ($\frac{5}{16}$ in.) wire was wound in the same manner as on the concrete tube. The two ends of the wire were fastened to a special apparatus by means of which an increasing tension up to the elastic limit could be imparted to the wire. In several points of the wire extensometers were fixed, and from the readings of the latter it was clear that no decrease in tension could be observed at any point in the wire, not even diametrically opposite to the point of application of the tension. In order to facilitate the transmission of the tension by reducing the friction between wire and concrete to a minimum the polygonal shape with twelve corners was chosen, and in each corner a small roller was inserted between the wire and the steel flat. In addition to these precautions the key on one wire was placed diametrically opposite to that of an adjoining wire.

(*b*) THE KEY.

Out of the great number of designs for keys to keep the ends of the wires together with the requisite amount of tension, six were actually made and tested with a view to ascertaining that no weakening of the area of the wire was caused by the fixing of the key. The design that afforded the simplest and quickest method of manufacture, combined with the lightest weight, was chosen.

(*c*) TIGHTENING APPARATUS.

The function of this fitting is to grip the ends of the wires and draw them together so as to stretch the wire gradually up to the elastic limit to start with, or slightly beyond it, because this results in a valuable increase of the latter. For this preliminary stretching of the wire no measuring of the force applied was necessary, as it was easy to observe when the elastic limit was reached, the mill-scale dropping off the wire at this point. This procedure served mainly to stretch the wire to the proper shape to fit the polygonal tube, after which the tension was relaxed in order to start afresh and impart the exact amount of tension required for the test ; after the first wire is stretched to the correct tension it is easy to stretch the remainder to the same stress by tuning them to the same pitch.

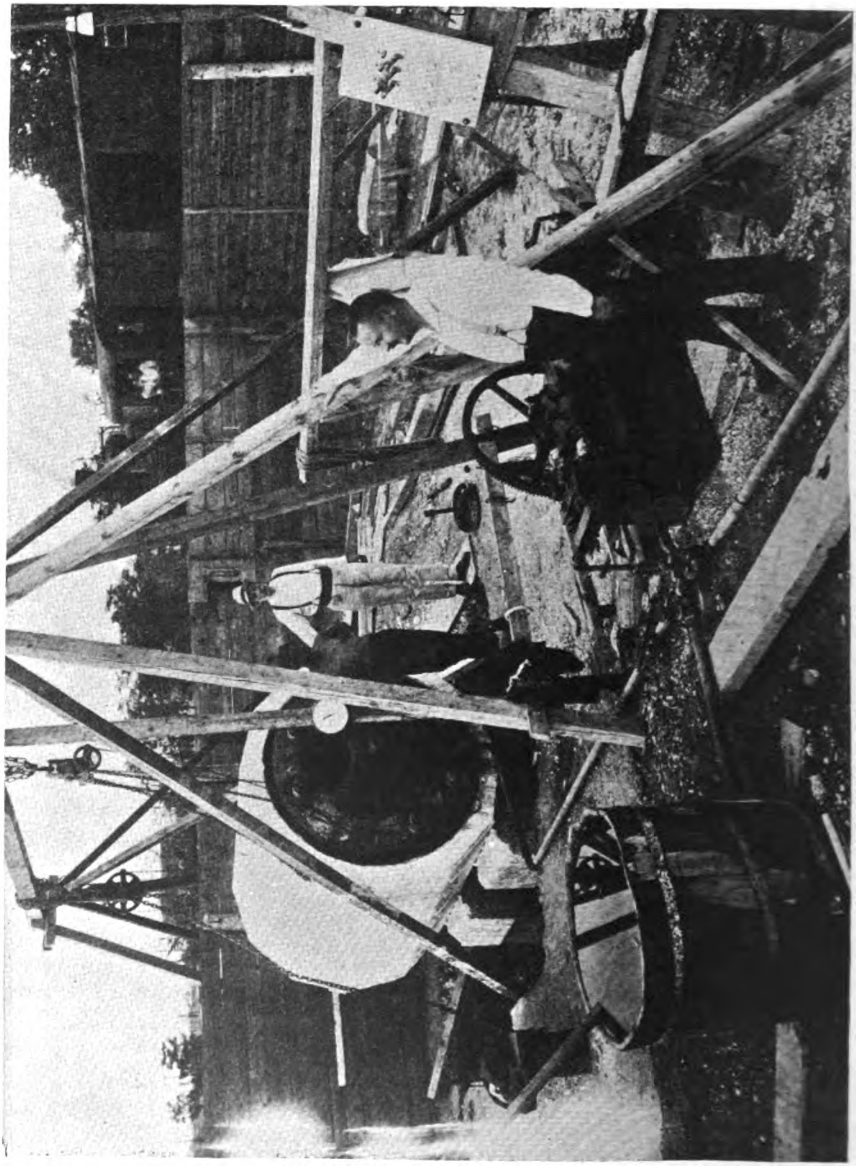


Fig. 7.

The most practical way to impart the necessary tension to the wire is to do so hydraulically by means of a small hand pump ; financial considerations unfortunately prevented the use of such an apparatus, which would undoubtedly have rendered possible the greatest accuracy in the stressing of the wire. Lever arrangements were tried at first, but finally it was decided to use an apparatus designed by Mr. Umlauf, which has the advantage of being easily handled. By this arrangement the two ends of the wire are gripped by a spindle with square threads so that the ends are pulled together by screwing until the desired tension is obtained ; when the latter is reached a slip action takes place so that the tension is kept constant, and in this position the key is fixed to the ends of the wire. In *Fig.* 6 both the tightening screw and the keys may be seen, the wires behind the screw having been stretched and keyed, whereas the ones in front of the screw are still loose and awaiting the tightening process.

(*d*) MATERIALS.

The choice of materials is generally left open, and the best results will, of course, be obtained by using first-class materials only. For the preparation of the test tube local materials only were available, such as were used for the reinforced concrete tubes, and the quality, more particularly in regard to the reinforcement, was hardly as good as might have been desired.

The concrete was mixed and placed in the same manner as for the other tubes for the Blumau power station, these being ordinary reinforced concrete pipes where the reinforcement prevents the tamping of the concrete, with the result that the tensile strength of the latter was comparatively low. In spite of the better facilities for producing a higher grade concrete in the test tube, either by means of efficient tamping or, better still, by the use of the cement gun, the same method of concreting was adopted as for the Blumau tubes, it being considered that the tensile strength of the concrete in question is only a factor in the total resistance of the tube, and the tests were mainly concerned with establishing the relation between the two ; an increase in tensile strength produces an increase in the total resistance. The tensile strength was found experimentally by means of two test beams, under the assumption that the pure tensile strength, which is the only factor that is of interest in the tube tests, is about half of the tensile strength in bending. In this way the ultimate tensile strength was found as 215 lbs. to 270 lbs. per sq. in., the lower value being taken as the basis for calculation.

The reinforcement consisted of ordinary mild steel of an ultimate tensile strength of 23–26 tons per sq. in. with an elastic limit of 19 tons per sq. in., which latter was raised by stretching to about 21 tons per sq. in. The wire was 8 mm. ($\frac{5}{16}$ in.) in diameter and was placed at 4·2 cm. (1⅝ in.) centres, corresponding to an area per inch length of tube of 0·047 sq. in. The effective thickness of the tube being 10·7 cm. (4¼ in.), the percentage *k* was about 1·1 per cent. On account of the low elastic limit it was not possible to use an initial stress of more than 23,000 lbs. (10 tons) per sq. in., or a tension in the individual wire of 1,600 lbs., corresponding to a tension *T* per inch length of tube of 970 lbs.

(*e*) OBSERVATIONS.

From equation (16) the calculated ultimate strength of the tube is found as

$$1{\cdot}5P = p'r = c_u t + c_u mA + m_o c_o A$$

or

$$p'r = 215 \times 4{\cdot}2 + 215 \times 45 + 0{\cdot}047 + 60 \times 1{\cdot}1\% \times 970 \quad . \quad . \quad (31)$$

$$p'r = 32p' = 900 + 460 + 640, \text{ from which}$$

$$p' = 28 + 14{\cdot}5 + 20 = 62{\cdot}5 \text{ lbs. per sq. in. or } 4{\cdot}4 \text{ atm.} \quad . \quad . \quad (32)$$

The actual ultimate strength revealed by the test was 60 lbs. per sq. in., or 4·2 atmospheres, at which pressure a longitudinal crack appeared close to the supports ; the agreement between the calculated and the actual figures was thus quite satisfactory. Had the ultimate tensile strength of the concrete been 355 lbs. per sq. in., an ultimate pressure of 90 lbs. per sq. in., or 6·5 atmospheres, might have been expected, and the allowable working pressure *p* in the two cases would be 42 lbs. and 60 lbs. per sq. in. respectively. After first letting off the water-pressure the latter was repeatedly increased from zero until leaking began, which invariably happened at a pressure of 37 lbs. per sq. in. (2·6 atmospheres). Equation (29) gives the value of m_o in this case as

$$m_o = \frac{pr}{kT} = \frac{37 \times 32}{1\cdot1\% \times 970} = 125 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (33)$$

This result is therefore also in perfect agreement with the figures found during the tests of the smaller tubes.

From equation (32) it will be seen that the pressure at which the initial compression should be overcome is 20 lbs. per sq. in., based upon $m_o = 60$. The crack, however, never opened until a pressure was reached corresponding to $m_o = 125$.

ECONOMICAL CONSIDERATIONS.

In practice the usual materials of construction for high-pressure tubes have hitherto been either steel or reinforced concrete. A comparison between the latter materials and the new method may be useful for drawing conclusions as to the economical value of the latter.

For low pressures the reinforced concrete tube was considered advantageous. The dimensions are determined by equation (2)

$$2\cdot5P = p'r = c_u(t + mA) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (34)$$

As the usual practice has been to take the value of m (in tension) as 20, this will be assumed throughout as a basis, although the higher value of 45 has been used during the experiments. During the tests on the large tube it was found that the surplus strength, due to the concrete itself, was represented by the difference between 4·2 atmospheres and 2·6 atmospheres, or 1·6 atmospheres (23 lbs. per sq. in.). The ultimate tensile strength of the concrete c_u found from this figure when taking $m = 20$ as basis is only 140 lbs. per sq. in. as compared with the value 215 lbs. per sq. in. found by test beams. With regard to the factor of safety 2·5, it is considered that this should be adhered to although in practice a factor of 2 is often considered sufficient. This gives a working load P for the two qualities of concrete, 215 lbs. and 355 lbs. per sq. in. ultimate strength respectively.

$$P = 86(t + 20A) \text{ or } 145(t + 20A)$$

or for a reinforcement percentage k of $2\frac{1}{2}$ per cent., in which case $t = 40A$

$$P = 86 \times 60A \text{ or } 145 \times 60A$$

$$P = 5{,}200A \text{ or } 8{,}700A \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

For high pressures the steel tube is generally preferred for several reasons. The latest steel practice uses an average working steel stress of only 13,000 lbs. per sq. in., inclusive of a 50 per cent. addition for joints. The factor of safety, having regard to welds, etc., is generally given as 1·5. The working load P is therefore expressed by

$$P = pr = 13{,}000A \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

In endeavouring to compare these two materials of construction it will simplify matters if the assumption is made that the cost of moulds, concreting, and internal rendering balances the precautionary measures against corrosion and the extra cost of transport of the steel tubes, so that the basis price of the steel may be taken as being the same in both cases, and a comparison can therefore be made through the weight of steel employed. A comparison between equations (35) and (36) makes it clear that the steel tube will always appear somewhat cheaper than the reinforced concrete tube owing to its utilising the steel better. Only in cases of low pressures where the steel tube cannot, for practical reasons, be reduced below a certain minimum does the comparison seem in favour of reinforced concrete. Even for low pressures it is not usual or feasible, in the case of steel tubes, to have a smaller thickness than $\frac{3}{16}$ in. for smaller tubes or $\frac{5}{16}$ in. for larger sizes, whether constructionally required or not, whereas in the case of reinforced concrete the steel area can be reduced *ad libitum*. If the allowance for protection against corrosion is brought into the calculation as $\frac{1}{16}$ in., so that the effective steel area for the smaller and the larger tube is $\frac{1}{8}$ in. and $\frac{1}{4}$ in. respectively that may be fully utilised, the working load P is found from

$$P = pr = 0\cdot125 \times 13{,}000 \text{ or } 0\cdot25 \times 13{,}000$$

for the smaller and the larger tube respectively. If the diameters are 12 in. and 64 in. respectively, the working pressure is found in the two cases as

p = 270 lbs. per sq. in. (19 atm.) and
p = 100 lbs. per sq. in. (7 atm.).

The comparison between steel and reinforced concrete becomes, of course, more favourable to the latter for lower pressures than the above, as exemplified by the comparative estimate below with a working pressure of 3 atmospheres. It may, however, be taken for granted that in a great many cases reinforced concrete will prove the more economical material in competition with steel.

When considering the third alternative, the initially stressed concrete tube, it is necessary to alter the m values in the same manner. By reducing m to 20, m_o may be increased correspondingly and may be expressed as

$$m_o = \frac{1}{k} = 100.$$

That this agrees well with the test results is shown in the following schedule which is based on equation (16) modified slightly as below,

$$p' = \frac{t}{r} c_u + m \frac{A}{r} c_u + m_o \frac{k}{r} \mathrm{T}.$$

In the tests with the smaller tubes, with c_u = 285 lbs. per sq. in. this becomes

$$p' = \frac{1 \cdot 2}{6} \times 285 + 20 \frac{0 \cdot 012}{6} \times 285 + 100 \frac{0 \cdot 01 T}{6}$$

And for the three values of T

$T = 282$; $p' = 57 + 11 \cdot 4 + 47 = 115 \cdot 4$ lbs. per sq. in. (8·2 atm.)
$T = 564$; $p' = 57 + 11 \cdot 4 + 94 = 162 \cdot 4$ „ „ (11·5 atm.)
$T = 1410$; $p' = 57 + 11 \cdot 4 + 235 = 303 \cdot 4$ „ „ (21·5 atm.)

whereas the corresponding figures for the larger tube, with c_u = 145 lbs. per sq. in. and T = 970 lbs. are

$$p' = 19 + 4 \cdot 3 + 33 = 56 \cdot 3 \text{ lbs. per sq. in. (4 atm.)}$$

As the actual test loads were in the four cases 8, 12, 18 and 4·2 atmospheres the agreement between the calculation and the experiments seems satisfactory also in this case.

In order to make a fair comparison of the relative costs of the three methods of construction, viz., steel, reinforced concrete and initially-stressed concrete, the assumption of a somewhat better concrete for the latter may be justified, in view of what has been mentioned in the description of the test tube ; for this reason c_u is taken as 215 lbs. per sq. in. instead of 145 lbs. per sq. in., whereby the ultimate water pressure becomes about 70 lbs. per sq. in. (5 atmospheres), and the working pressure may be taken as 43 lbs. per sq. in. (3 atmospheres). The tests repeatedly proved that the initial stress accounted for 37 lbs. per sq. in. (2·6 atmospheres), so that in the case under consideration the concrete would only be called upon to account for the remaining 6 lbs. per sq. in. (0·4 atmospheres), which would only produce a tension in the concrete of 35 lbs. per sq. in., equivalent to a factor of safety of 6 against cracks.

As a basis for the comparison of the three cases is taken a tube with a working pressure of 28 lbs. per sq. in. (2 atmospheres) and provision against a 50 per cent. overload, or an allowable pressure of 43 lbs. per sq. in. (3 atmospheres), which produces a tension in the walls of the tube of

$$P = pr = 43 \times 32 = 1{,}375 \text{ lbs. per in. length.}$$

(1) STEEL TUBE.

The calculated thickness of the tube is

$$t = \frac{1{,}375}{13{,}000} = 0 \cdot 106 \text{ in.}$$

The addition of $\frac{1}{16}$ in. as protection against corrosion increases the thickness to

0·1685 in.; for tubes of this diameter a minimum of ⅜ in. must, however, be taken. The weight of steel is, therefore, per inch length of tube

$$0{\cdot}375 \text{ in.} \times 64 \text{ in.} \times \pi \times 0{\cdot}285 \text{ lb.} = 21{\cdot}5 \text{ lbs.}$$

(2) Reinforced Concrete Tube.

If the tube is reinforced with 2·5 per cent. of steel, the calculated thickness of the tube, reckoning with an allowable tension in the concrete of 85 lbs. per sq. in., is found from

$$P = 1{,}375 = 85(t + 20 \times 0{\cdot}025t) = 127t$$

as $t = 11$ in. and $A = 0{\cdot}27$ sq. in.

The weight of steel is therefore

0·27 in. × 75 in. × π × 0·285 lb.	=	18·0 lbs.
add 12½ per cent. for longitudinals	=	2·2 lbs.
Total		20·2 lbs.

It will be seen that for this case there is a possibility of making the reinforced concrete tube at the same price as the steel tube; further reduction in price is only possible by cutting the factor of safety.

(3) Initially-Stressed Tubes.

In the test tube the effective thickness of the tube was 4¼ in., which, by adding a further inch of protective rendering, is brought up to 5¼ in. The weight of the steel is

0·047 in. × 75 in. × π × 0·285 lb.	=	3·2 lbs.
add 50 per cent. for keys and longitudinals	=	1·6 lbs.
Total		4·8 lbs.

From these figures it will be evident that the tube uses about half the amount of concrete and between one-third and one-quarter of the steel. It may further be pointed out that the test was carried out with ordinary mild steel wire, so that the use of a good high-tensile steel wire would make the test show up still better, the price of the latter being about double but the strength nearly four times that of mild steel.

A comparison between the costs of high pressure tubes may not be out of the way for the sake of completeness, as in this case the steel is fully utilised. An allowable working pressure of 145 lbs. per sq. in. (10 atmospheres) is taken as a basis, and produces a tension in the tube walls of

$$P = pr = 145 \times 32 = 4{,}650 \text{ lbs. per in. length.}$$

(1) Steel Tube.

The thickness is found as

$$t = \frac{4{,}650}{13{,}000} = 0{\cdot}36 \text{ in.,}$$

which, with the addition of $\frac{1}{16}$ in., gives 0·42 in. This gives a weight of steel

$$0{\cdot}42 \text{ in.} \times 64 \text{ in.} \times \pi \times 0{\cdot}285 \text{ lb.} = 24 \text{ lbs.}$$

(2) Reinforced Concrete Tube.

At this pressure this method of construction can no longer be recommended.

(3) Initially-Stressed Tube.

Assuming $c_u = 285$ lbs. per sq. in., $k = 1$ per cent. and an initial steel stress of 80,000 lbs. per sq. in., the thickness of the concrete is found from

$$1{\cdot}5P = c_u t + mc_u A + m_o c_o A$$

$$\text{or } 1{\cdot}5 \times 4{,}650 = 285t + 20 \times 285 \times \frac{t}{100} + 100 \times 800 \times \frac{t}{100}$$

$$\text{or } 7{,}000 = 1{,}142t;$$

$$t = 6 \text{ in.}$$

It is seen that the initial stress of 800 lbs. per sq. in. in the concrete, giving a total compression per inch length of tube of

$$T = 800 \times 6 = 4{,}800 \text{ lbs.},$$

is not even neutralised by the working load of 4,650 lbs., so that no tension stresses at all appear in the concrete, which is an important fact in connection with the water-tightness.

The weight of the steel is

0·06 in. × 76 in. × π × 0·285 lb.	=	4 lbs.
add 50 per cent. for keys and longitudinals	=	2 lbs.
Total		6 lbs.

In dealing with still higher pressures it would no doubt be of advantage to employ a thin sheet steel lining, and a combination of this system with the initially stressed concrete promises very satisfactory results. This combination is being tested at present and the results will be published shortly.

A summary of the tests mentioned in this article is given below in tabulated form so as to facilitate comparison :—

SUMMARY OF TESTS.

Number of Tests.	Internal Diameter in inches.	Thickness in inches.	Reinforcement		Initial tension per wire in lbs.	Initial Steel Stress S_0	Elastic Limit.	Initial concrete stress C_0	Ultimate test pressure p'	Repeat pressure p_0	m_0	$p_1 = p' - p_0$	C_u calculated with $m = 20$.
			per cent.	diam. in inches.									
2	35	7	2·25	$\frac{1}{2}$	0	0	—	0	140	0	—	140	250
2	28	10	2·25	$\frac{5}{8}$	0	0	—	0	268	0	—	268	260
2	12	1·2	—	—	—	—	—	—	57–85	0	—	57–85	285–425
1	12	1·2	1	$\frac{5}{64}$	112	23,000	100,000	240	112	56	120	56	285
1	12	1·2	1	$\frac{5}{64}$	224	46,000	100,000	490	170	100	105	70	350
1	12	1·2	1	$\frac{5}{64}$	560	115,000	185,000	1,200	255	210	90	45	215
1	64	4·2	1·1	$\frac{5}{16}$	1,600	23,000	41,000 (48,000)	265	60	37	118	23	140

Concrete Buildings at Markham Colliery.

THE concrete buildings at Markham Colliery, near Doncaster, illustrated in our last issue, were constructed of blocks made on the well-known "Winget" machines, manufactured by Messrs. Winget, Ltd., of 24, Grosvenor Gardens, London, S.W.1.

THE "DUO-SLAB" SYSTEM OF COTTAGE CONSTRUCTION.

In view of the fact that over 3,000 cottages have been built on the "Duo-Slab" system, and that the Edinburgh Town Council has now decided to carry out its large new housing scheme by this method of construction, the following particulars and the accompanying illustrations should be of particular interest at the present time when, as pointed out in our Editorial pages, it seems that the 1923 Housing Act is being rapidly adopted by local authorities. The "Duo-Slab" system is the invention of Messrs. Wm. Airey & Son, Ltd., of Greek Street, Leeds, who have erected cottages on this system in Leeds (1,600 cottages), Newcastle-on-Tyne (311 cottages), and elsewhere. The head of the firm, Sir Edwin Airey, by the way, was elected Lord Mayor of Leeds at the annual meeting of the Leeds Corporation last month.

The system has but little in common with the better-known methods of concrete wall construction, most of which are either forms of monolithic construction or, by the use of concrete blocks, follow the principles of brickwork. It is essentially a system for the mass production of houses on a unit system, although the unit can be varied to suit any design. It is by mass-production that cheapness is obtained, however, and if the architect and contractor worked in close collaboration it should be possible to build a very large number of houses using the same unit of construction—in this case the size of the slab—without monotony. On the Leeds scheme the "unit" is 4 ft.—the distance between the centres of the piers—and as will be seen from the illustration of the house before roughcasting (*Fig.* 9) this lends itself admirably to door and window openings.

A further advantage of this system is that it does away with the necessity for the employment of that *rara avis* the bricklayer: we know the trade unions of this craft claim that concrete block-laying should be done by their members, but they will have considerable difficulty in convincing one that the laying of slabs one on top of the other, while the trowel still rests in the store, is the legitimate field of the bricklayer.

The houses may, of course, be finished with roughcast, cement rendering, or any desired method.

Briefly, the wall is formed of pre-cast slabs laid on edge one above another with a vertical space between each tier of slabs which is filled in with concrete

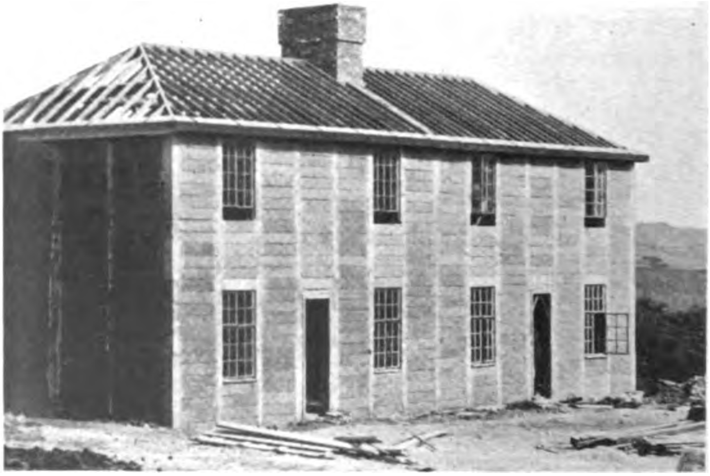


Fig. 9.—Duo-Slab Houses before Roughcasting.

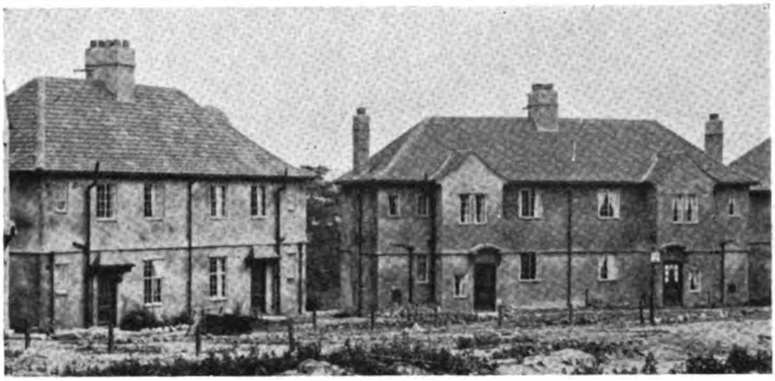


FIG. 10.—CONCRETE HOUSES AT LEEDS.

poured *in situ*, thus binding all the slabs together at their ends.

MOULDING THE SLABS.

Although the slabs may, of course, be of any desired size, in the cottages illustrated (which are all on the Leeds housing scheme, for which Mr. H. S. Chorley was the architect), they are 3 in. thick by 8 in. high. The ends are splayed in order to get a stronger bond at the piers, and the length of the slabs is 3 ft. 8 in. on the longer face and 3 ft. 6 in. on the shorter face (*Fig.* 5,p. 799). The piers measure 8 in. across the face of the wall, and thus occur at 4 ft. centres all round the cottages. The slabs are formed of a concrete consisting of 5 parts hard clinker, 1 part sand, and 1 part Portland cement.

An interesting patent mould is used for the manufacture of the slabs. As shown in *Figs.* 1, 2, 3 and 4 (p. 798), the mould consists of a wooden frame hinged at three corners and having fastenings at the other corner, so that it can be stood up on edge to enclose a rectangular area a little deeper than the thickness of the slab (so that there will be sufficient depth for the concrete necessary to form the slab before it is consolidated) or opened out in order to remove it from the finished slab. The area enclosed by the slab is the length of the slab and sufficiently wide to enable three slabs to be formed, face up, side by side and separated by longitudinal runners or division plates. A plan of the mould is given in *Fig.* 1 ; *Fig.* 2 is a side elevation ; *Fig.* 3 an end elevation showing one of the hinged corners and the corner which is locked to keep the mould in position ; *Fig.* 4 is a side elevation showing three moulds placed one above another (the hinged corners are on the right and the locked corners on the left). The vertical alignment of one mould above the other is ensured by two iron straps on each face which project downwards and pass over the face of the mould immediately below ; or, alternatively, by pins fixed in the bottom of the mould entering holes in the upper edge of the mould below.

The method of tamping is also a novel one. This operation is performed by placing over the top of the mould a board with a rebated edge. The rebates project over the top of the mould, so that when pressure is applied the board (except for the rebates) is pressed into the concrete until the rebates come to rest on the edges of the mould, thus ensuring uniformity in the thickness of the slab. The pressure is applied by a man stamping his feet on the board, or by striking the board with a rammer.

After the first three slabs have been made in the bottom mould the tamping board is removed and the concrete covered with paper or sprinkled with sawdust—Messrs. Airey prefer the latter method, as it absorbs the excess moisture from the concrete and also roughens the surface sufficiently to give a good key for the external roughcast or rendering. Another mould is then placed on the first, and a fresh batch of concrete poured, tamped, and covered with sawdust, and this operation is repeated until a height is attained beyond which it is inconvenient to work, usually about 12 moulds. The forms are

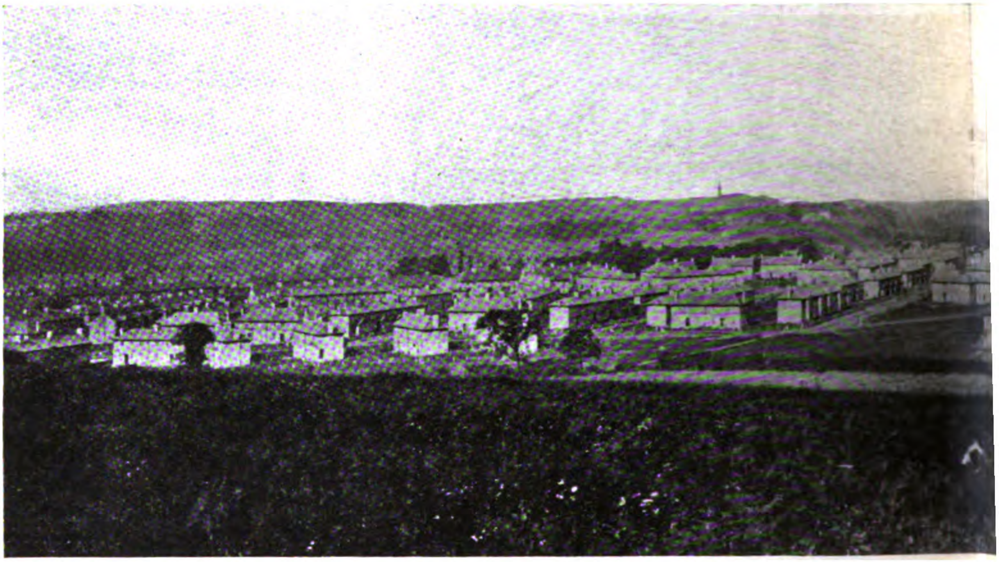


Fig. 11.—General View of the

removed the following day and used again, and the slabs left to cure. The removal of the moulds is an extremely simple matter, all that is necessary being to unfasten one corner and walk round the pile of slabs "unfolding" the mould away from the slabs. By this method the slabs are all made quite close to the site where they are to be used (*Fig*. 13, p. 797, which shows the slabs alongside the footings), and the unnecessary labour often entailed by carrying blocks on pallets away from the moulds to a curing ground and back again to the site of the work is eliminated. We are informed that with the use of these moulds it is possible for one trained labourer to make 100 slabs per day, and as cottages of the type being built by local authorities require on an average 1,000 slabs, it therefore takes ten days for one man to make sufficient slabs for one house.

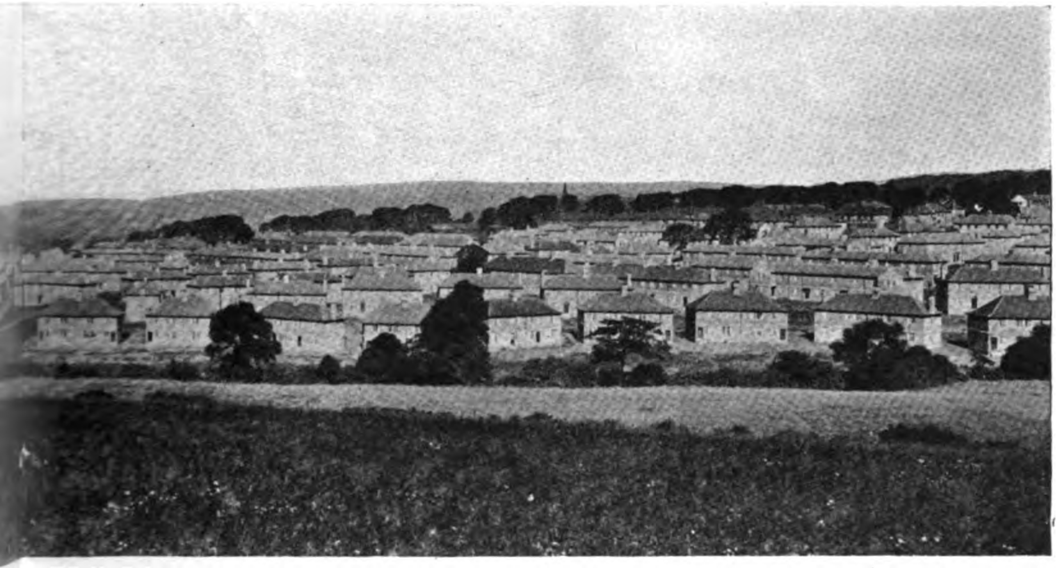
METHOD OF CONSTRUCTION.

The walls are formed of two leaves, with a 2-in. cavity between, making a total thickness of 8 in. The blocks are of the same size for both leaves, and no variation is made in the concrete for the external and internal walls.

The method of erection is to place the slabs on edge one above another without horizontal mortar joints, strips of wood, which also form the sides of the piers and are afterwards removed, being used as distance pieces. The distance between the slabs along the outside of the wall is 6 in., and on the inner face 4 in., the difference being accounted for by the chamfered edges (see *Fig*. 5). As the slabs are laid shuttering is erected to enclose these spaces on the outer faces of the two leaves forming the wall, and pieces of wood are placed inside the cavity 3 in. from the ends of the slabs. Within this shuttering concrete is poured *in situ*, forming a solid pier throughout the height of the wall at 4-ft. centres.

CONSTRUCTION OF PIERS.

In the construction of these piers an ingenious patented system of shuttering is employed (*Figs*. 6, 7, 8, p. 799). When the slabs (*a*, *a*) have been placed in position the uprights (*f*, *f*′) of the framework are first placed in their vertical position as at *Fig*. 6, that is, on the outer surface of the two rows of slabs, horizontally across the space between the edges (*e*, *e*), and with cross pieces (*j*, *j*) transversely across the two rows of slabs forming the wall. The uprights of the framework are locked in a plumb position by the upper transverse loops (*l*, *l*) and the screw shaft (*q*). The wedges (*u*, *u*) act as distance pieces and also retain the slabs in position



[Mr. H. S. Chorley, Architect.

NCRETE HOUSING SCHEME AT LEEDS.

and press the outer surface of the latter against the portions of the uprights below the pivot bolt (*k*). When the concrete which forms the pier has been run it is allowed wholly or partially to set, and when it has become hard enough to permit of one or both the uprights being removed, another set of concrete slabs is placed upon the preceding ones, and after the framework has been released by the withdrawal of the wedges it is raised and readjusted to the new position, when the operation of forming the pier is continued until the wall is carried to the required height. *Fig.* 6 is a plan showing the shuttering in position, with a pier shown in section ; *Fig.* 7 is a front elevation of the shuttering ; *Fig.* 8 is a side elevation of the apparatus with the slabs shown in section. This system ensures that the walls are built true, and the metal facing to the wooden framework results in a smooth finish.

Fig. 12 (p. 796) shows the “Duo-Slab” system of walling in course of erection.

A general view of the concrete houses at Leeds is given in *Fig.* 11, while some of the cottages on this estate are shown in *Figs.* 10, 14 and 15 (Frontispiece).

Letters Patent have been granted to Messrs. Wm. Airey & Son, Ltd., for this special form of concrete house construction ; the Company, however, has already granted licences to several contractors and are prepared to issue licences to other approved contractors for the use of the system in the erection of houses on royalty terms which have been approved by the Ministry of Health.

“DUO-SLAB” HOUSES AT SWANSEA.

A few days ago the Swansea Housing Committee gave further consideration to the question of concrete houses for the Maesteg site, and after some discussion decided to accept, subject to the approval of the Council, the tender of Messrs. Wm. Airey and Sons, Ltd., for 200 “A” (non-parlour) Type and 50 “B” (parlour) Type houses on the “Duo-Slab” system.

In correspondence read from the contractors it was stated that in the construction of these houses 90 per cent. of the labour would be unskilled, and only 10 per cent. skilled, and that the whole of the work would be completed approximately in one year.

The Borough Architect (Mr. Ernest Morgan) stated that the prices were reasonable.

The Chairman (Mr. Percy Molyneux) stated that the houses built on this system, which he had inspected at Leeds, were very good and their construction would help them to get back to a semi-normal position in housing.

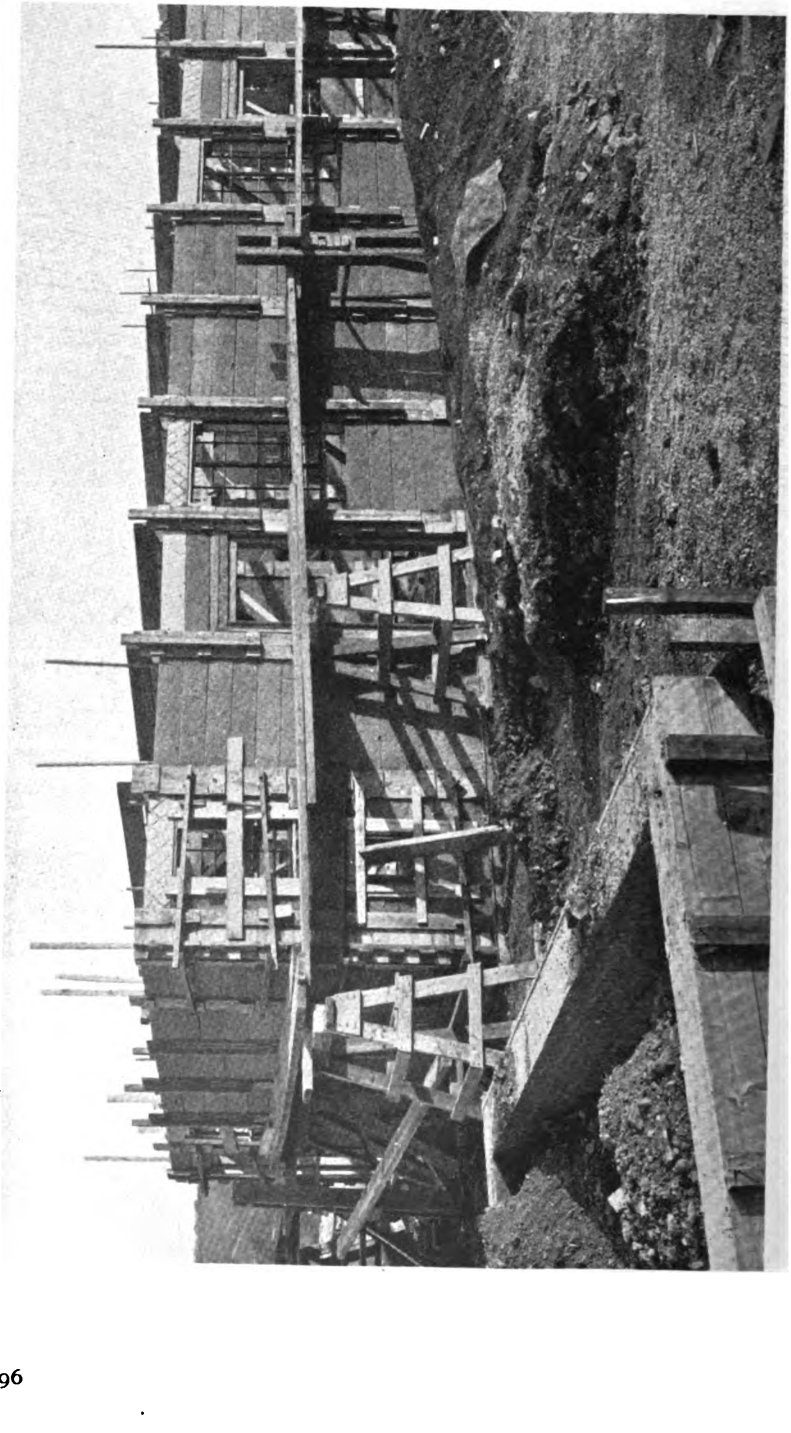


FIG. 12.—"DUO-SLAB" HOUSES IN COURSE OF ERECTION.

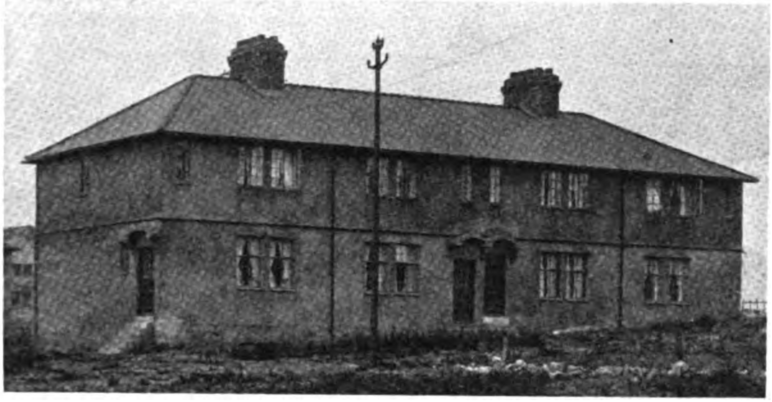


FIG. 14.—CONCRETE HOUSES AT LEEDS.

A letter was read to the effect that s far as labour was concerned there would be no difficulty in the erection of houses.

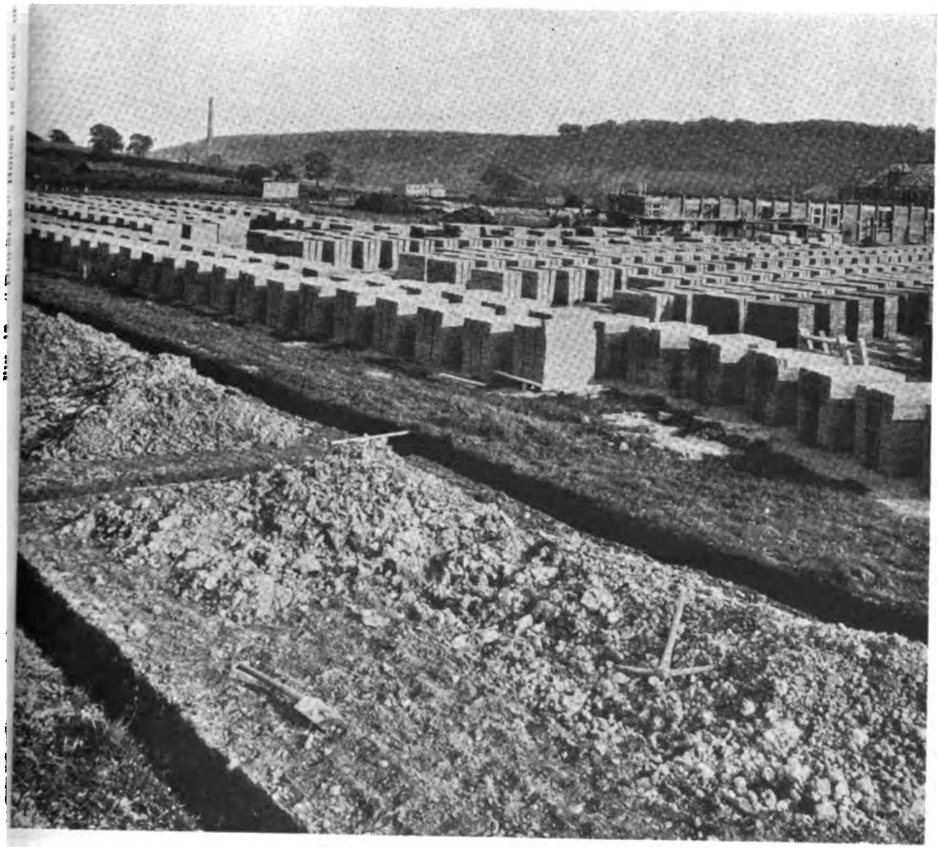
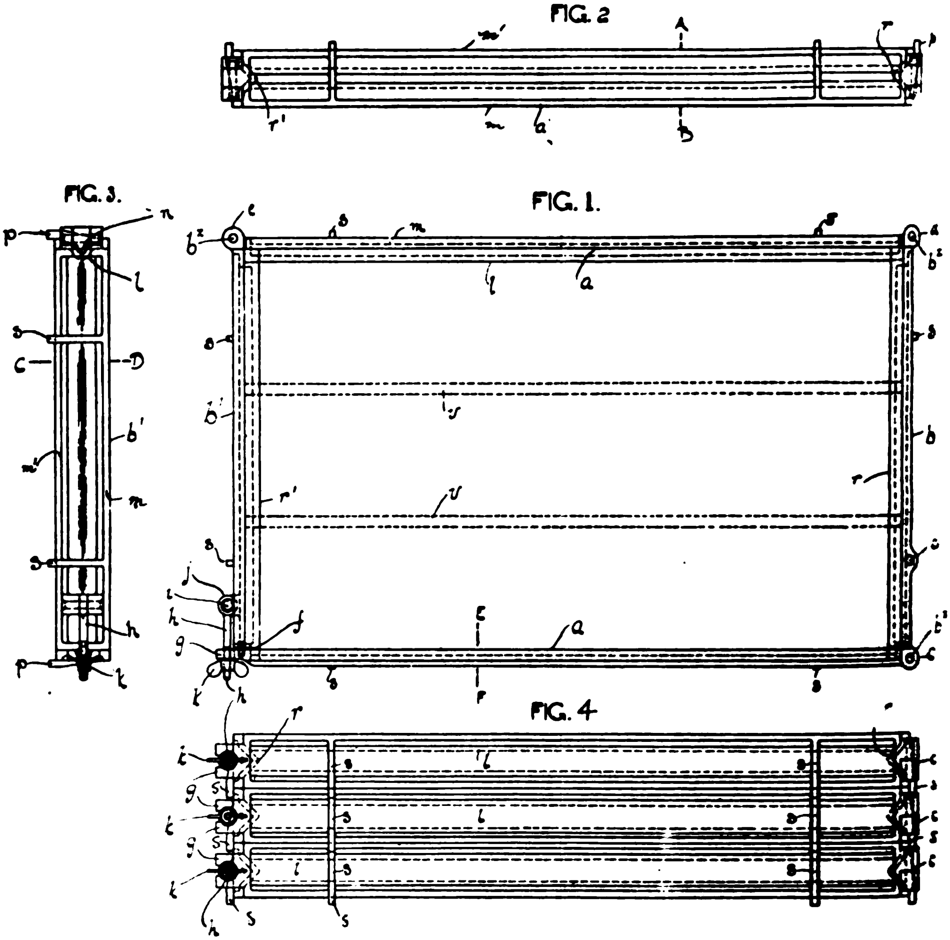


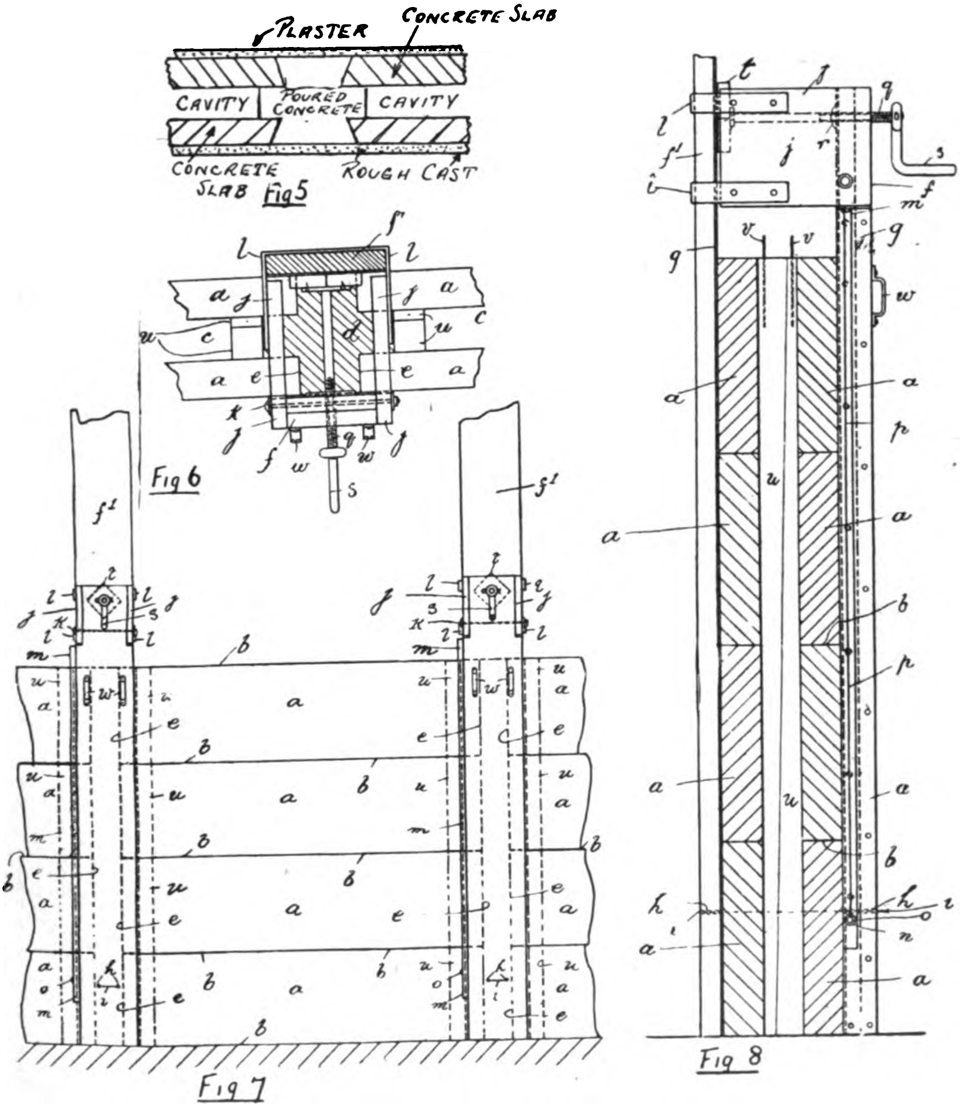
FIG. 13.—SLABS CURING CLOSE TO SITE OF HOUSES.



Moulds for Making Slabs.

Fig. 1, Plan; Fig. 2, Side Elevation; Fig. 3, End Elevation; Fig. 4, Side Elevation showing three Moulds placed one above the other.

THE "DUO-SLAB" SYSTEM OF COTTAGE CONSTRUCTION. (*See p. 792.*)



Shuttering for Forming Piers.

Fig. 5, Horizontal Section of Finished Wall; Fig. 6, Plan showing Shuttering in Position; Fig. 7, Front Elevation; Fig. 8, Side Elevation with Slabs shown in Section.

THE "DUO-SLAB" SYSTEM OF COTTAGE CONSTRUCTION. (*See p.* 792.)

THE STRENGTH OF SHORT CONCRETE BEAMS.

By J. B. MACPHAIL, B.A., B.Sc., A.M.E.I.C.

THE design of reinforced concrete beams loaded on spans of less than four times their depths has received little attention either theoretically or experimentally, and the experiments herein described were undertaken in the hope that they might furnish some information that might be of use to designers.

BEAMS TESTED.

The elements of the beams tested are given in *Table* 1. The beams were all 6 in. in breadth and 6 in. longer than the spans listed. All rods had hooked ends and were made from ordinary commercial round bars; in every case three rods were placed with their centres 1½ in. from the bottom and 1½ in. from centre to centre and from the sides of the beam. When five rods were used, the additional two were placed with their centres 3 in. from the bottom and covered the spaces in the lower layer. The sand used had a fineness modulus of 2·68; 93 per cent. passed a No. 14 sieve; 34 per cent. passed a No. 28 sieve; and 6 per cent. passed a No. 48 sieve. The aggregate was ¾ in. limestone chips containing some dust. The concrete was hand-mixed in proportions of 2 : 4 : 6 with sufficient water to give a slump of 4 to 6 in. in a 6 in. by 12 in. cylinder. Water was poured over beams and test cylinders once a day from the fourth to the twenty-eighth day after mixing.

TABLE No. 1.

Mark.	Reinf.	Depth in.	Span in.	Safe Load from B.M. W1.	v.	u.
A	5 × 9/16	10¼	17	16,250	205	139
B	3 × 5/8	10¼	27	10,600	121	124
C	3 × ½	10¼	36	7,080	78	100
D	3 × ½	10¼	45	5,660	63	80
F	3 × 9/16	12	33	11,250	105	119
G	3 × ¾	12	44	7,630	70	90
H	5 × ¾	16	28	23,300	164	125
K	3 × ¾	16	43	11,000	72	91

Using unit stresses of 750 lbs. per sq. in. on concrete and 16,000 lbs. per sq. in. in the steel, values of *W*1, the safe central concentrated loads were calculated. In order to keep bond stresses down to reasonable values all but beams *G* and *K* were over-reinforced; so that in these two the resisting moment of the steel was less than that of the concrete. In the others the concrete resisting moment governs. For these loads the unit shear *v* and the bond stress *u* were calculated.

The beams were tested at an age of 56 days under a central concentrated load on the Emery testing machine in the Strength of Materials Laboratory at McGill University.

RESULTS OF TEST.

The observed ultimate loads *W*3 are given in *Table* 2 together with safe loads *W*2 based on a unit shear of 56 lb. per sq. in., a usual value for beams without web reinforcement. *W*3 ÷ *W*1 give factors of safety *F*1, and *W*3 ÷ *W*2 give factors of safety *F*2. This method of presenting results is due to Dr. Oscar Faber.

Results obtained by using the loads observed at the first drop of the indicator of the testing machine gave figures of the same character as those tabulated.

In general failure occurred through diagonal cracks starting at or near the supports and running up to a point beneath the load; sometimes by vertical cracks below the load.

TABLE No. 2.

Mark.	Safe Shear W2 lbs.	W3 lbs.	F1.	F2.
A	4,430	56,000	3·4	12·6
B	4,880	44,400	4·2	9·1
C	5,060	25,600	3·6	5·1
D	5,060	17,600	3·1	3·5
F	6,000	39,600	3·5	6·6
G	6,070	31,000	4·1	5·1
H	7,950	98,300	4·2	12·4
K	8,570	33,800	3·1	3·9
		Mean	3·65	7·3

The observed ultimate compressive strength of four 6-in. by 12-in. test cylinders at 8 weeks averaged 3,080 lbs. per sq. in., so the observed average factor of safety requires a reduction to the basis of an ultimate strength of 1,875 lbs. per sq. in. at 4 weeks, 40 per cent. of which was taken as the working stress. To form a rough estimate, assume that the ultimate load carried by a beam is proportional to the ultimate strength of the concrete of which it is made. The ultimate loads for the weaker concrete would then be

$$W_3 \times \frac{1875}{3080}$$

and the observed factors of safety F_1 would be reduced in the same ratio, or

$$F_3 = F_1 \times \frac{1875}{3080} = 2{\cdot}22 \text{ average,}$$

which agrees fairly well with the assumed factor of safety of

$$\frac{1875}{750} = 2{\cdot}50.$$

Further, the value of the denominator 3,080 in the above expression should be reduced slightly to bring it to its value at 28 days. A value of 2,740 lbs. per sq. in. as the ultimate strength of the test cylinders at 28 days would give a factor of safety F_3 of 2·50, and an increase of 11 per cent. gives 3,040 lbs. per sq. in. The probability is that the increase in a further 28 days is more than 11 per cent., so the true strength at 28 days is probably lower than 2,740 lbs. per sq. in. This would slightly increase the factor of safety from 2·50. It may be mentioned that the tentative report of the Commission du Béton Armé de l'Association Belge de Standardisation contemplates an increase in strength of 50 per cent. between 28 and 90 days.* In any case the reduced factor of safety F_3 is not far from the value of 2·50 assumed.

This revision of concrete stress is applicable also to the factors of safety F_2, but this would not alter the fact that the values of F_2 vary over a much wider range than values of F_1.

The small variation in factors of safety F_1 for the design loads in which bending and bond stresses only were considered, and the high values of the unit shear neglected, show that the concrete was economically used, whereas a design based on a maximum unit shear of 56 lbs. per sq. in. would waste concrete in some cases, as shown by the unnecessarily high factors of safety F_2, though a little steel might be saved.

SHEAR STRESSES.

To explain the action of the shear stresses reference may be made to the works of Dr. Oscar Faber, who has developed a theory of the existence in a loaded reinforced concrete beam of a series of direct and indirect inclined compressions which, together with the steel, resist the shear. This theory is given briefly in *Reinforced Concrete Design* by Faber and Bowie, vol. 1, p. 79 ff., and at greater length in his paper "The Shearing Resistance of Reinforced Concrete Beams"* read before the Concrete Institute in London in May, 1916.

The application of Dr. Faber's methods to this particular type of beam and loading gives safe loads, based on shearing stresses, that are identical with those determined from considerations of bending, and the uniformity of the factors of safety F_1 confirm his deduction that in a simple beam having reinforcement with sufficiently hooked ends running the whole length at a distance from the bottom not less than $\frac{1}{3}$ kd, failure will not occur until the full moment of resistance has been reached, and that if the safe bending stresses are not exceeded, the safe stresses elsewhere will not be exceeded.

It may be argued that the use of a greater amount of steel to reduce bond stresses than would be required to resist the applied bending moment caused the ultimate loads on the beams so treated to be higher than they would have been if the bond requirements could have been met by steel of a smaller total area, because of the smaller unit stress and the consequent smaller deformation of the steel. This can be positively decided only by experiment, but the theory does not indicate such a result, and practical considerations of bar spacing and construction would doubtless prevent much alteration in this direction where short spans are concerned.

It may be concluded then that short beams simply supported, if designed for safe moment and bond stresses, do not require web reinforcement.

Acknowledgment is tendered to McGill University for facilities for testing the specimens; to Professor Ernest Brown, M.E.I.C., and Mr. S. D. MacNabb, A.M.E.I.C., for making the tests, and to Mr. J. A. McCrory, A.M.E.I.C., for facilities for making the specimens and for assistance in testing them.

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* *Pratique du Calcul du Béton Armé*, by G. Magnel, p. 128.

* See *Concrete and Constructional Engineering*, May, 1916, and subsequent numbers.

"THE CONCRETE YEAR BOOK, 1924."

THE large number of inquiries we are continually receiving for names of makers of different specialities, addresses of firms and engineers, etc., have convinced us that there is a great need for a complete directory of the many branches of the concrete industry. It is, in fact, an anomaly that a growing industry which has already reached such large proportions should be without its directory, and we have decided to fill this gap by the publication early in January next of the first edition of "The Concrete Year Book." This volume will be a Handbook and Directory for the Concrete Industry, and should be of considerable value to all interested in the many branches of that industry.

The first part of the book will consist of special articles dealing with such subjects as the economical use of cement, essentials in the production of good concrete, proportioning and grading of aggregates, concrete floors, concrete roads, concreting in cold weather, and other aspects of the uses of concrete. A specially-prepared series of tables for reinforced concrete design will also be included. Another feature will be a bibliography of all books dealing with reinforced concrete and allied subjects.

The remainder of the book will contain a complete Directory of the industry, classified under different headings, including Consulting Engineers who Specialise in Reinforced Concrete, Reinforced Concrete Designing Firms, Public Works Contractors who Specialise in Concrete Construction, Concrete Flooring Specialists, Suppliers of Aggregates and Sand, Artificial Stone Manufacturers, Manufacturers of Colouring Materials for Concrete, Manufacturers of Machinery and Plant for Concrete and General Constructional Work, Manufacturers of Concrete Products, Manufacturers of Reinforcement, Contractors for Concrete Roads, Manufacturers of Proofers and Hardeners for Concrete, Piling Contractors, Manufacturers of Asbestos-Cement Products, Concrete Testers, etc. No pains have been spared to make the Directory Section as complete and accurate as possible.

The volume, which will be strongly bound, will be published by Concrete Publications, Ltd., Publishing Department, 4 Catherine Street, Aldwych, W.C.2, price 3*s.* post free, and copies may now be ordered by filling in the form on *p.* 822.

CONCRETE TANKS FOR FUEL OIL.

THE large extent to which concrete is used for the storage of fuel oil in America is shown by an article in a recent issue of a daily contemporary dealing with the oilfields of California. "One of the most remarkable developments of the great oil production of California," it is stated, "is the concrete reservoir. The reservoir is made by scooping out an area several acres in extent, lining the sides and bottom with three inches or thereabouts of concrete and putting a wooden cover over it. It is a storage place for fuel oil. The crude, as it comes from the field, is run through the stills to take off the lighter products, which would be lost by evaporation. These lighter products go into storage in steel tanks as refinery stock for future reference. On the tank farm of the General Petroleum Corporation, near Watson siding, about five miles from the ocean docks at San Pedro, one of these reservoirs recently completed holds 1,750,000 barrels, and it was filled within 30 days. In addition, the same company has two other concrete reservoirs holding 500,000 barrels each, besides a whole string of steel tanks to hold the lighter products, and has another immense concrete tank in its plans. The Standard Oil of California is building at its refinery at El Segundo a concrete reservoir covering 14 acres which will hold 2,225,000 barrels, and is about to start on another one which will be much bigger than even this, probably 3,000,000 barrels capacity. The General Petroleum tank farm covers 72 acres. In the same vicinity, at Watson Siding, lies the main tankage field of several of the big companies, covering about 600 acres. Here various companies are building storage which will hold 20,000,000 barrels.

CORRESPONDENCE.

CONCRETE FLOORS AND FIRE RISKS.

Sir,—As you know, the Fire Office Regulations oblige reinforced concrete designers to make floor slabs not less than 5 in. thick, and a reinforced concrete floor slab constructed in this manner must be calculated in accordance with the L.C.C. Regulations for reinforced concrete if the building is in the London area. There are on the market at present a number of so-called hollow floors, the strength of which depends either partially or totally upon steel bars embedded in concrete; or, in other words, small members in reinforced concrete.

In spite of this, this particular form of constructing floors, which is admitted by the Fire Office on condition that the total thickness of the hollow floor amounts to 5 in., is not subject to the regulations of the L.C.C. for reinforced concrete.

This appears to me a gross injustice to all reinforced concrete designers, who are, of course, obliged to apply the stringent regulations of the L.C.C., whereas the makers of various patent hollow floors are allowed to provide whatever reinforcement they think is necessary without being subjected to any regulations whatever as far as calculations are concerned.

Is it not time that these hollow floors should be subject to certain regulations with regard to the calculation of their strength, especially when it is obvious that many of them are nothing less than a form of reinforced concrete? Until something of the kind is done the designer of monolithic reinforced concrete floors is bound to be at a disadvantage when competing against various patent hollow floors.

I am under the impression that when the Fire Office regulations were drafted the minimum thickness of 5 in. was only meant to apply to a reinforced concrete floor slab having this thickness, and not to a hollow floor having two thicknesses of, say, 2½ in. each, which is obviously much less fire-resisting than an unbroken thickness of 5 in.

This, of course, is a matter which concerns the Fire Office and the insurance companies, whereas in my opinion the other question concerns the L.C.C.

G. C. Workman,
Managing Director.
Edmond Coignet, Ltd.

WATERPROOFING A LEAKY FLOOR.

Sir,—The following may interest those of your readers who wish to render concrete watertight. For the present I have cattle under a concrete floor that leaked like a sieve. One dressing of cement-wash well worked in with a broom has rendered this floor quite impervious. For the last month it has been covered with water ½-in. deep in places, whilst men have been moving heavy stones thereon, yet the under surface has remained bone dry.

C. H. Cooper
(*late Borough Engineer, Wimbledon*).
Gugh St. Agnes,
Isles of Scilly.

Carbon Dioxide and the Hardening of Concrete.

Although the increase of strength with age in cubes made of sand and cement is mainly due to hydrolysis, Goslich and Hart have recently found that it is also partly due to the action of carbon dioxide on the free lime in the mass, and to increase in density due to the glue-like action of the gelatinous material which Michaelis first found to be produced when a cement began to set. The increase in strength is greater when the concrete is cured for part of the time in air and part in water than when it is immersed in water during the whole of the curing period, because with the combined curing the gelatinous material can dry and harden, whereas when the concrete is immersed in water this gelatinous material remains in a swollen and spongy state. The action of the carbon dioxide is confined almost wholly to the surface of the concrete, as it does not penetrate more than one-third of an inch into the mass.—*Zement.*

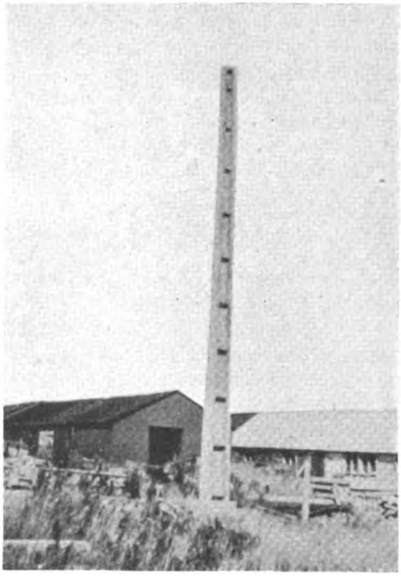
CONCRETE POSTS AND FENCING.

THE two photographs of fencing show the first attempts made at concrete fencing many years ago by Mr. W. Marriott, M.Inst.C.E., Engineer and Manager, Midland and Great Northern Railway, who writes :

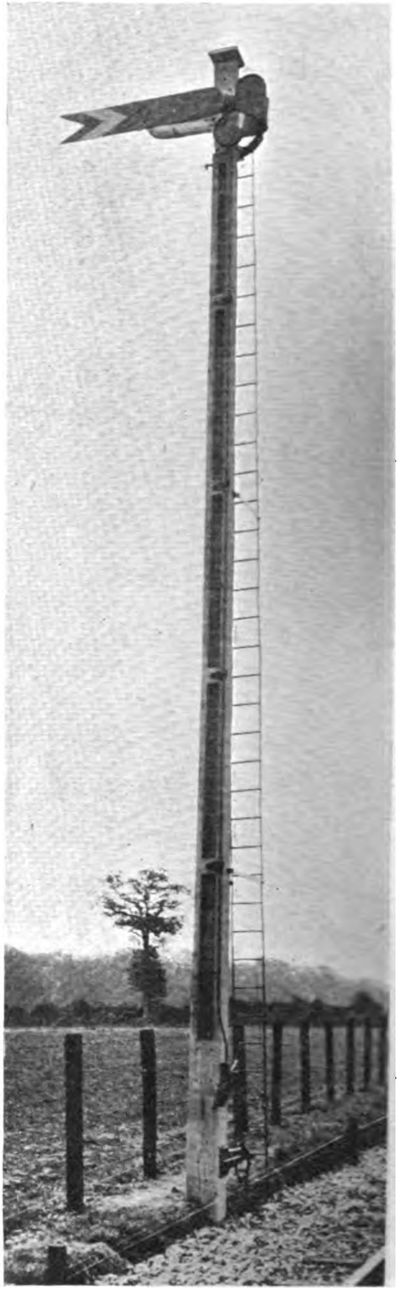
" The line fence represents the forerunner of hundreds of miles of similar design, as this design is being used elsewhere. The platform fence and lamp-post, which were made in much fear and trembling, have also stood for the best part of twenty years and are likely to stand many times twenty years.

" My reason for showing the old fencing is that concrete is comparatively new in this country. I suppose I am the only person that has got a bridge built entirely of concrete fifty years old, and therefore it is very necessary to get into people's minds that concrete is durable if properly made. Unfortunately, I know people who have had badly-made articles, improperly reinforced, that have failed, and one failure does the business an enormous amount of harm.

" Following on this the photograph of a large ' H '-section power-pole, some 32 ft. high, recently made to take a pull of 3,000 lbs. for a double-circuit 3,500-



CONCRETE POWER-TRANSMISSION POLE.

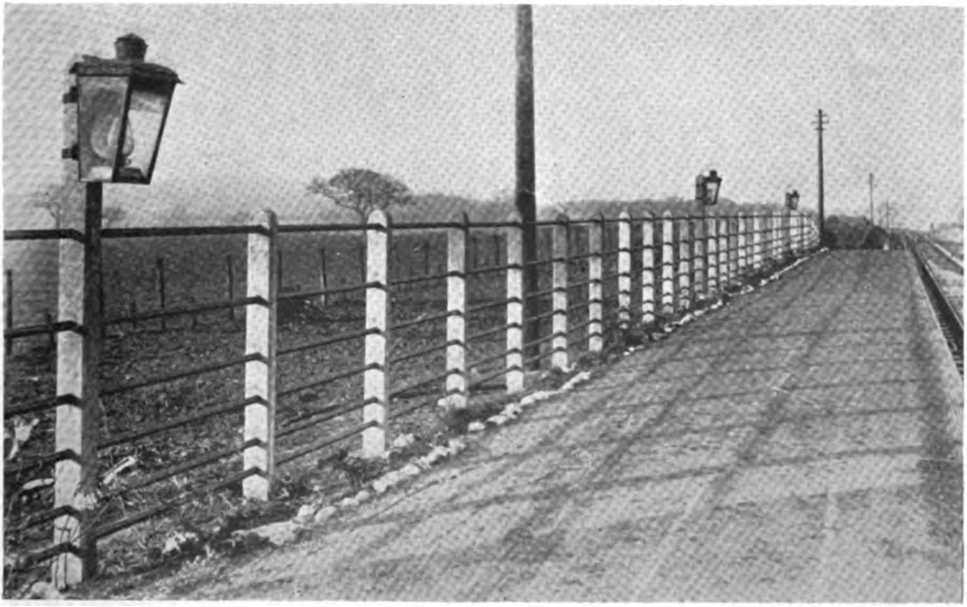


"H" SECTION CONCRETE SIGNAL POST.

volt transmission line, believed to be the biggest concrete power-pole made in England, is likely to be interesting. It is a far cry from fence-posts to power-poles, yet the same principles govern both, i.e., strength and durability. It may be mentioned that the 'H'-section power-pole was made by the Abdon Clee Stone Quarry Co., of Ditton Priors, and is a very fine piece of work.

"The photograph of a new design of signal post (registered), 'H'-section, believed to be the first signal post of this section, also shows an advance in the same direction. This signal post was made at Melton Constable and erected near by."

Concrete Line Fence.



Concrete Platform Fence.

QUESTIONS AND ANSWERS RELATING TO CONCRETE.

***Readers are cordially invited to send in questions relating to concrete. These questions will be replied to by an expert, and, as far as possible, answered at once direct and subsequently published where they are of sufficient general interest. Readers should supply full name and address, but only initials will be published. Stamped envelopes should be sent for replies.*—Ed.**

STRESSES IN T-BEAMS WITH COMPRESSION STEEL.

Question.—In the design of T-beams we are told that when the neutral axis lies in the slab the rectangular beam formula should be used, and *vice versa.* In Thomas and Nichols' design tables, T-beams have their neutral axis in all cases in the web, therefore a T-beam not included in these tables must be looked for in the tables for rectangular beams. In Hool's *Concrete Engineer's Handbook* and Warren's *Engineering Construction,* Vol. II, it is not clear as to what procedure is to be followed in order to tell whether the neutral axis is in the slab or web.

I take it that in using these formulæ it is necessary to calculate by one formula, and if, say, the formula for the neutral axis in the web is used and gives a result showing the neutral axis to be in the slab, a recalculation by the other formula must be made.

In *Reinforced Concrete Design,* by Faber and Bowie, I am aware that the whole question is immediately simplified and solved on page 40, but so far as I am aware this is the only text-book in which this point has been elucidated. In any case I see no advantage in not reverting to first principles in all cases, regarding the steel area as n times its area of concrete.

Take, for instance, the beam shown on page 47, *Concrete and Constructional Engineering,* Vol. XVIII, No. 1.

With $N - 15\ A'_s = N \times 12{\cdot}2 = 183$ sq. in.

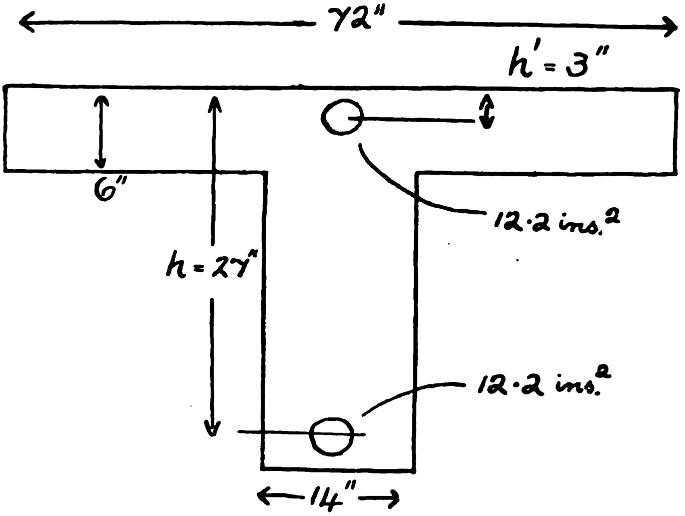
$$A_s = 183 \text{ sq. in.}$$

Then

$$(58 \times 6)\left(\frac{6}{2} + Y - 6\right) + \frac{14Y^2}{2} + (183)(Y - 3) = (183)(27 - Y).$$

$$7Y^2 + 714Y - 6{,}534 = 0,$$

from which $Y = 8{\cdot}4$ in. and not 9·7 as stated in the reply.



$$I - \frac{1}{3}\left(BmY^3 - (Bm - B)(Y - h)^3\right) + nA(h - Y)^2 + nA'(y - h')^2.$$

$$\frac{1}{3}\left(72 \times 8{\cdot}4^3 - (58)(2{\cdot}4)^3\right) + 183\,(18{\cdot}6)^2 + 183\,(54)^2$$

$$\frac{1}{3}\,(41{,}800 - 800) + (63{,}300)\,(5{,}335)$$

$$\underline{82{,}600}$$

$$F_c - \frac{My}{I} - \frac{4{,}600{,}000 \times 8{\cdot}4}{82{,}600} = 467 \text{ lbs./in.}^2$$

i.e. with $F_c = 600$ lbs./in.,^{2} $M = 5{,}900{,}000$ and not 4,600,000 in. lbs. given in the reply in your January number, nor 5,350,000 as given on page 348 of your May number.

$$F_s - \frac{nM(h-y)}{I} = \frac{15 \times 4{,}600{,}000 \times 18{\cdot}6}{82{,}600} = 15{,}540 \text{ lb./in.}^2$$

$$F_c \text{ steel} = \frac{nM = \dfrac{(y-h')}{(5{\cdot}4)}}{82{,}600} = 4{,}500 \text{ lb./in.}^2$$

Total compression taken by steel,
4,500 × 12·2 = 55,000 lbs.

Total tension taken by bottom steel
= 189,000 lbs.

Total compression taken by concrete, = (1,800 × 72) + (160 × 14).

i.e. area of compression triangle × width of flange = 131,840 lbs.

Total compression concrete and steel = 187,000 lbs. (not 250,000 as stated).

The tension steel is thus stressed to approximately the correct amount, whereas the concrete and compression steel is understressed. Is not this the correct result and correctly arrived at ?

If it is required to obtain the stress of 16,000 and 600 lbs. respectively with a B.M. of 4,600,000 in. lbs. and a flange width of 72 in. and a depth of 6 in., web thickness 14 in. and depth 27 in., referring to Thomas and Nichols' table the B.M. per foot width = 60,800 ft. lbs., area of steel 1·87 sq. in., whereas the B.M. per ft. width required is 64,000 ft. lbs.

B.M. for 28-in. depth = 64,300 ft. lbs. per ft. width of slab without any compression reinforcement, and it would therefore appear desirable to use a 28-in. beam without compression steel.

However, if 27 in. depth is the limit, then the outside B.M. to be taken care of by compression steel is, say, 26,000 ft. lbs., which at 3 in. from the compression flange and 27 in. depth of beam requires 2 sq. in. steel (Thomas and Nichols' tables).

The area of tension steel to balance this is

$$A_s = \frac{M}{F_s(d - d)} = \frac{26{,}000 \times 12}{16{,}000\,(27 - 3)} = \tfrac{13}{16} \text{ in., say } \tfrac{3}{4} \text{ in.}$$

The steel would be :

Tension (1·87 × 6) + (¾ in.) = 11·97 sq. in. = 12 sq. in., say, 10 1¼ in. rods.

Compression = 2 1⅛ in. rods.

Checking this design by calculation, we have :

$$(58 \times 6)\left(\frac{6}{2} + Y - 6\right) + \frac{14\,Y^2}{2} + 30\,(Y - 3)$$

$= (184)\,(27 - Y)\ 348(Y - 3) + 7Y^2 + 30\,Y - 90 - 4{,}968 + 184Y = 0.7Y^2 + 562Y - 6{,}102 = 0$ and $Y = 9{\cdot}68$ in.

$$I = \frac{1}{3}\left(72 \times 9{\cdot}7^3 - 58 \times 3{\cdot}7^3\right) + 184\,(17{\cdot}3)^2 + 30\,(6{\cdot}7)^2$$

$$\frac{1}{3}\,(65{,}712 - 2{,}937{\cdot}7) + 54{,}427 + 1{,}346.$$

$$I = 76{,}700.$$

$$F_c = \frac{M}{I} \times Y = \frac{4{,}600{,}000}{76{,}700} \times 9{\cdot}7 = 60 \times 9{\cdot}7 = 582 \text{ lbs./in.}^2$$

$$F_s = \frac{M}{I} \times N(H - Y) = 900 \times 17^3 = 15{,}570 \text{ lb. ins.}^2$$

$$F_c \text{ steel} = \frac{Mn}{I} \times (Y - H') = 900 \times 6{\cdot}7 = 6{,}030 \text{ lbs.}$$

This method appears so simple and straightforward, and requires no complicated formulæ to remember or a textbook to refer to, that it appears strange that it has not been more universally adopted, as it is all that is required (knowing the various B.M.'s of the beam) to complete the calculation.

As a rule, the tables would be used. The problem is usually more the design against diagonal tension than against the direct tensile strains, and on this subject theory must give way to practice.—P. F.

Answer.—We are unable to follow the

equations or calculations given early in the question, as no explanations are given. We would, however, point out that our correspondent finds the steel stress under a moment of 4,600,000 in. lbs. comes to 15,540 lbs./in., which agrees well with the very simple calculation given on p. 50, January, 1923, in which 16,000 was taken as a basis.

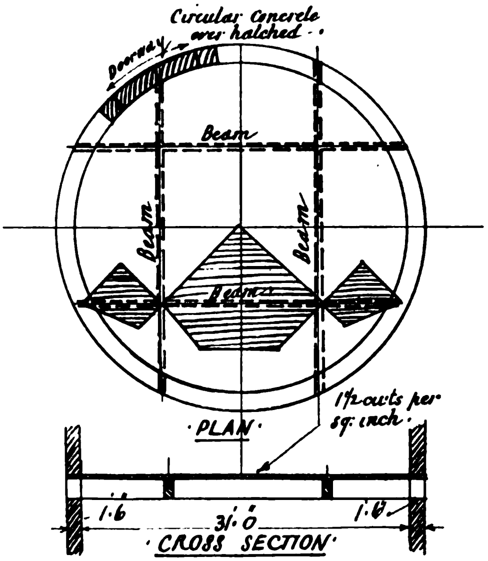
It is therefore quite wrong to write the moment of resistance down as 5,900,000 in. lbs., since with this greater moment the steel in tension would be grossly overstressed.

We are still convinced that the original calculation in our January issue is by far the simplest, and is substantially accurate.

O. F.

TANK FLOOR.

Question.—A circular concrete floor, 31 ft. diameter inside the 18 in. supporting brick walls, is to be carried by four concrete girders as shown in the sketch. The floor is to sustain a live load of 1½ cwts. per sq. ft. in addition to the dead load. How should the slab and girders be designed? I suggest that as the centre panel is 15 ft. by 15 ft. the reinforcement should be laid in both directions.—T. C. G.



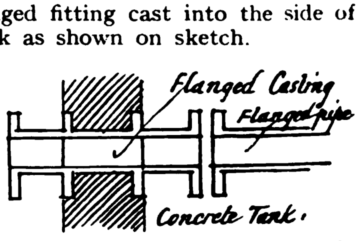
Answer.—Owing to the symmetry of the arrangement the four beams will deflect equally at their four points of intersection, and will, therefore, neither carry each other nor be carried by each other. The load on each beam will therefore be the area shown shaded on the sketch, from which the bending moment can readily be calculated.

As regards the slab, it would certainly be well to reinforce the centre panel in both directions symmetrically, whereas the four oblong panels at the sides could have the reinforcement from the centre panel continued right across to the support with a small number of cross rods at right angles thereto.—O. F.

CONNECTING PIPES TO CONCRETE TANKS.

Question.—Which is the correct way of connecting lead or wrought-iron pipes to a reinforced concrete rectangular tank or cistern? Is it necessary or convenient to apply the silicate of soda solution inside in order to guarantee or increase the impermeability? Can the solution of chloride of magnesium take the place of silicate of soda?—M. J. V.

Answer.—The best method of connecting pipes to reinforced concrete tanks is to provide them with flanged connections which may be bolted to a special flanged fitting cast into the side of the tank as shown on sketch.



The use of silicate of soda inside the tank is not necessary, but is recommended, and chloride of magnesium is not recommended as a substitute.—O. F.

PUBLIC WORKS, ROADS AND TRANSPORT CONGRESS AND EXHIBITION.

In connection with the Public Works, Roads and Transport Congress and Exhibition, held at the Royal Agricultural Hall, London, from November 22 to 29, a large number of papers on various aspects of the activities of local authorities were read under the auspices of different professional bodies. The following abstracts are from the addresses more particularly dealing with the applications of concrete. A review of the Exhibition is given on p. 815.

CONCRETE ROADS IN SURREY.

In the course of a paper on road reconstruction, Mr. W. P. Robinson, A.M.Inst. C.E., County Surveyor of Surrey, said he had recently entirely reconstructed and widened a main road carriageway with cement-concrete surface. The old road varied in width from 13 ft. to 20 ft. with a surface-tarred water-bound macadam crust, whilst the new carriageway was 30 ft. wide and the sub-crust and surface were entirely of Portland cement concrete 8 in. thick. In places the new surface had to be laid in half widths in order to keep the road open for traffic. No foundation was provided other than the shaping of the ground to a camber of 1 in 50; the sub-soil, of loamy sand, was well rolled before placing the concrete, which was placed in two coats simultaneously. The lower course was 6 in. thick of 4 parts gravel, 2 parts sand and 1 part cement, and the upper or surface layer was 2 in. thick of 4 parts gravel, 2 parts sand and 1½ parts cement. The gravel used was washed and screened to all pass a 1-in. ring and graded so that 60 per cent. passed a 1-in. ring and was rejected by a ½-in. ring; 40 per cent. passed a ½-in. ring and was rejected by a ⅛-in. ring.

The sand was screened and afterwards washed to give the following grading:—55 per cent. passing 10-mesh sieve and rejected by 30-mesh; 30 per cent. passing 30-mesh sieve and rejected by 80-mesh; 15 per cent. passing 80-mesh sieve and rejected by 100-mesh.

The concrete was laid in alternate bays not more than 35 ft. long by 20 ft. wide, and the reinforcement was generally 10 lbs. per sq. yd., but the weight was varied according to the stability of the sub-soil. The joints between bays were laid at an angle of 60 degrees to the kerb line and the reinforcement was laid parallel to and at right angles to the joints, which were vertical butt joints afterwards filled with hot bitumen. Two mixing machines were employed, one for the lower course concrete and one for the upper course, so that the concrete could be laid as one course and operations on any one bay continued without cessation until that bay was finished. The concrete after levelling off to the finished surface was immediately covered with waterproof covers for about six hours, when the covers were removed and the surface of the concrete brushed with stiff brooms to remove all surplus scum; the surface was then immediately covered with 3 in. of wet sand, which was not removed for 28 days. Particular attention was given to the amount of water used, which was one of the most important points to watch in order to eliminate cracks. The correct proportion of water to give maximum strength in the concrete necessitated much more labour in ramming the concrete than had been expended on some of the concrete roads he had inspected. Cubes 6 in. in all dimensions made from concrete taken from the mixing machines and actually used on the road withstood on test a crushing load of 78 tons, or an average load of 4,870 lbs. per sq. in.; the cubes, however, were not reinforced in any way.

Before any surface dressing was applied to the concrete, traffic was permitted to use it for some weeks, so that any loose mortar on the surface might be disintegrated and swept off. When the surface showed a smooth, hard, mosaic appearance he preferred to apply a dressing of waterproofing material, and after experimenting with various substances he found that good results were obtainable by giving first a dressing of tar-oils followed by either hot or cold bitumen, which would adhere better than if applied direct to the concrete.

The concrete carriageway referred to, which was 1½ miles in length and 30 ft.

wide, had not yet shown any cracks, nor had the joints been chipped or affected in any way, although the road had been used almost daily for six months by a 14-ton traction engine hauling three trucks of gravel to the work. The surface was absolutely free from corrugation. The camber was 1 in 50, and curves of 900 ft. radius had been given a maximum super-elevation of 7 in. from kerb to kerb, which were of precast concrete in 3 ft. lengths, and which might be set back when a widening of the carriageway became necessary.

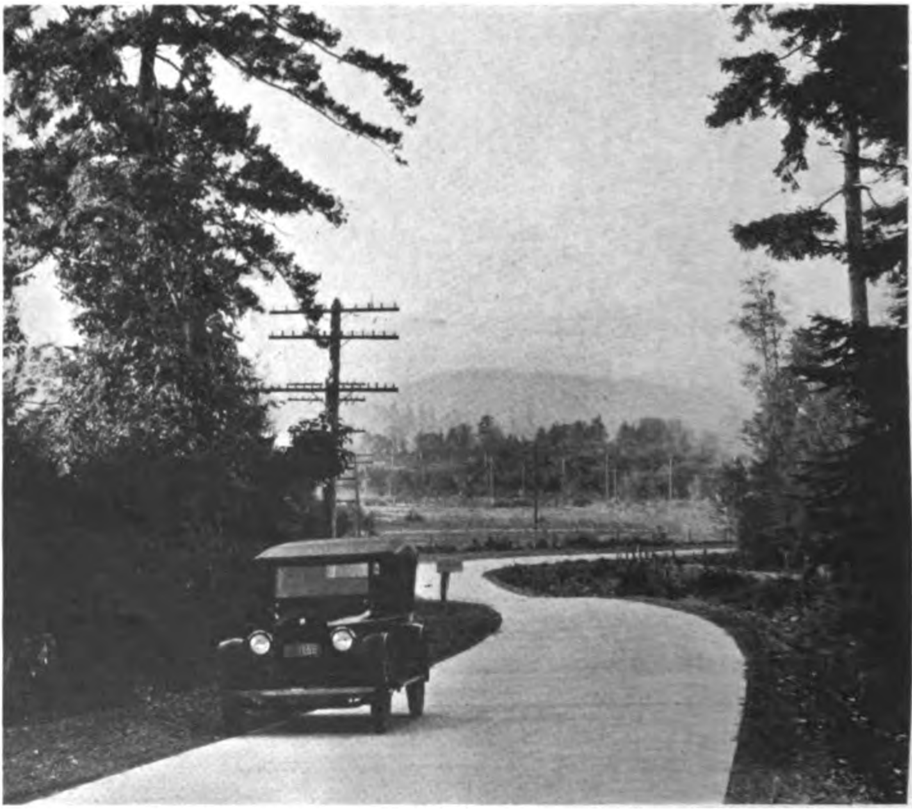
The kerbs had been made on the site in steel moulds specially made to his design (*see illustration*, p. 811), the object being to batter the roadside face in order to obviate abrasion of the top edge by traffic.

He adopted a concrete surface for the 30-ft. carriageway of the new arterial roads (15 miles) which were about to be constructed in the County of Surrey, as he was satisfied that an 8-in. layer of reinforced concrete, if properly laid and cured, would give greater satisfaction to the users of the road and be more economical in the end than a tarmacadam or bituminous carpet on a foundation of hardcore or pitched stone.

DISCUSSION.

In the ensuing discussion the Chairman (Sir Henry G. Maybury, K.C.M.G., C.B., Director-General of Roads, Ministry of Transport), asked Mr. W. P. Robinson if he would like to make any addition to his paper before the discussion took place.

Mr. Robinson said he wished to add one or two interesting points that had, unfortunately, been omitted from his



East Saanich Road, British Columbia.
A CANADIAN CONCRETE ROAD.

paper. The period of curing the test block had not been given; this was twenty-eight days. Another important factor missed was that the concrete roads referred to were entirely constructed by unskilled labour and by ex-service men. With regard to the reinforcement used in one mile of road he had varied the reinforcement ten times.

The chairman, in concluding the meeting, spoke on the great part that transport service played in the life of the community, and said there was no doubt that it was one of the key industries of the country. The construction of new roads helped more to solve the housing problem than almost anything else. He also referred to the need for economy; and pointed out that what was necessary for Hyde Park Corner and the Mansion House was not essential for the highlands of Scotland and Wales. He had taken out some figures in connection with the cost of roads, and had found that if only one penny could be saved on each yard of classified roads, £1,800,000 could be saved in one year; he hoped all the delegates would do their utmost to affect these economies that were so badly needed.

ROAD FOUNDATIONS.

Mr. Ernest Worrall, M.Inst.C.E. (Borough Surveyor of Stretford), gave an interesting address, entitled "Road Foundations and Carpeting: A Review."

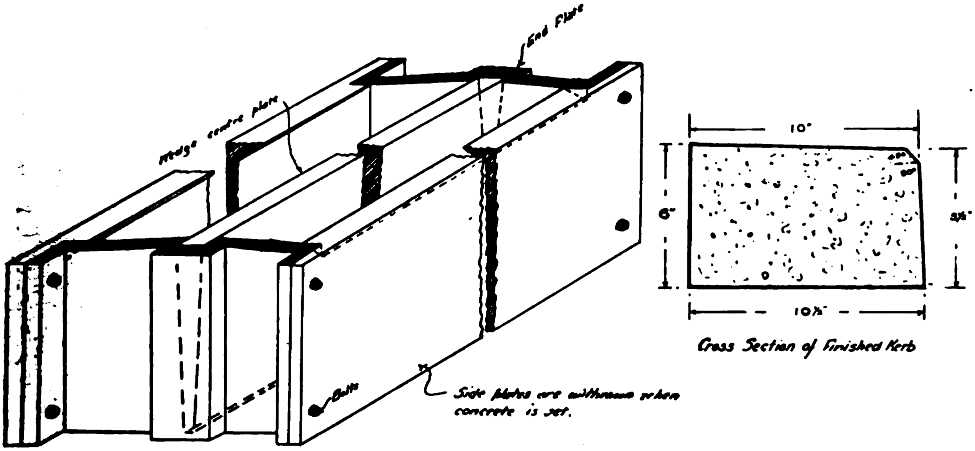
In dealing with the question of foundations, he expressed the opinion that present and prospective requirements would undoubtedly best be met by replacing existing foundations with cement concrete, and that that material must sooner or later be generally adopted for the purpose. The universality of its chief components, its adaptability, the comparative ease with which it could be made and used, its reliability given reasonable care in the selection, proportion and quality of its ingredients, and its feasible cost, all stamped the material as the future road foundation.

Given, he said, a normally good concrete and carpeting of, say, 8 in. in depth, and a wheel load plus impact of 6 tons on a 4-in. tyre, if 1 in. of the circumference were in contact and the spread of pressure were assumed to be contained within angles of 45°, the pressure on the underbed, as was clear from Diagram 1 (p. 812), was reduced to 50·8 cwts. per sq. ft.—a pressure that experiment had shown could be sustained without movement by firm ground.

$$\left(\frac{6 \times 20 \times 144}{20 \times 17} = 50{\cdot}8\right)$$

The unit of pressure on the underbed was proportionately reduced by an increase in the width of tyre; one 6 in. wide, *e.g.*, on similar data, reduced the load to 46·2 cwts. per sq. ft.

$$\left(\frac{6 \times 20 \times 144}{22 \times 17} = 46{\cdot}2 \text{ cwts.}\right)$$



Cross Section of Finished Kerb

MOULD FOR TWO LENGTHS OF CONCRETE KERB SIDE BY SIDE, AS USED BY MR. W. P. ROBINSON, A.M.INST.C.E. (COUNTY SURVEYOR OF SURREY), ON THE SURREY ARTERIAL ROADS.

Diagram 2 showed the curve of pressure due to varying widths of tyre under the above conditions.

Diagram 3, from the report of the U.S. Bureau of Public Roads, showed from experimental data the thickness of concrete slabs which fractured under the given wheel loads.

An important feature of cement concrete for road foundations was that there was no internal movement, and consequently no interstitial wear. *Per contra*, it had been calculated that in a ballast and macadam road the internal grind and wear amounted in a time proportionate to the pace and weight borne to as much as 30 per cent. of the whole. After filling the voids the surplus worked to the surface as dust in dry weather and mud in wet periods. The cost of removing that surplus and the contingent injury to vegetation and property in the vicinity by one or the other waste must in the aggregate represent a prodigious sum, a cost of bad roads which was apt to be overlooked, and one which if avoided must amount to a substantial contribution to the cost of a superior pavement.

The utility of rubble and macadam already on the site where foundations were being laid was considerable; carefully screened and separated from the dust and smaller particles such stone formed excellent aggregate.

The quantity of water used in the mixing of cement concrete deserved more than passing notice. It was no uncommon thing to see the mixture surcharged with water, especially in hand-mixing, either from a mistaken notion of benefit or for easier manipulation on the part of the workmen. Some recent experiments in Germany on the influence of the percentage of water content in high grade concrete gave the following results :

	Water %.	7 days.	28 days.	45 days.
Tension .	11	1	1	—
	17	0·870	0·825	—
	20	0·415	0·516	—
Compression	11	1	1	1
	17	0·536	0·603	0·673
	20	0·333	0·362	0·537

The figures showed that the addition of quantities of water largely in excess of 11 per cent. seriously reduced the strength of the concrete in the shorter periods, but that the effect diminished somewhat with time.

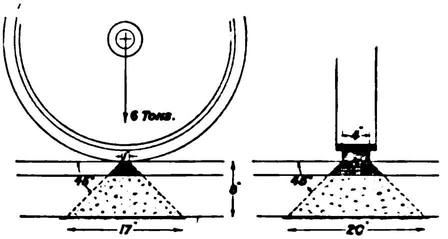


Diagram 1.

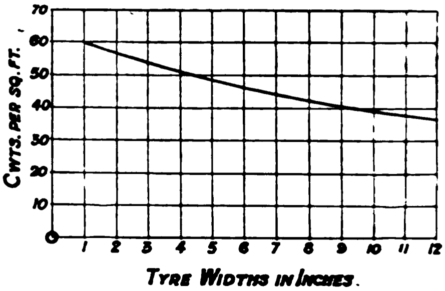


Diagram 2.

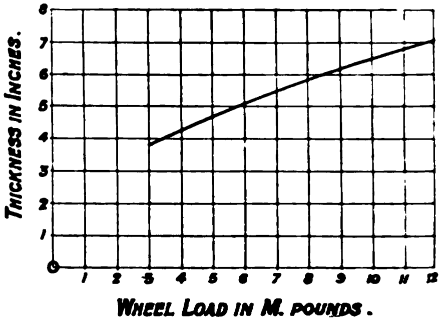


Diagram 3.

Protection of the concrete in the earlier stages of hardening or setting and drying likewise needed attention to ensure good results. The permeability and ultimate strength were directly affected by undue wetness and too rapid drying in ordinary Portland cement concrete. The series of tests referred to showed that slabs covered by sacks kept moist for two days and then placed under water for 28 days became impermeable to water at 120 lbs. pressure per sq. in. maintained for 4 hours. When allowed to harden in a dry atmosphere for the same period, ·86 lbs. pressure showed penetration of water in a half-hour test. Those tests proved the vital importance of shielding new concrete with a moist protective covering from too rapid drying by exposure to the sun and air. It was found also that imperviousness and strength were increased proportionately as the percentage of interstices or voids was reduced.

Experiments in America recently on the fatigue of concrete when used in pavements showed that, as in metals, there was a limit of endurance, even in good samples. Within 50 per cent. of the modulus of rupture a wheel load of 550 lbs. applied a million times produced no failure, but with the load increased to 780 lbs. failure under like conditions otherwise occurred with 5 per cent. only of that frequency of application. The modulus of rupture obtained in this elaborate series of tests in Illinois was determined at 720 lbs. per sq. in.

The comparatively new practice of gaining additional strength and resistance to static, running, and impact stresses in concrete road foundations, or alternatively securing adequate strength, by reinforcement in a reduced thickness of concrete was becoming almost general. Academic discussion as to the need for or utility of reinforcement in average or normal conditions was futile since there was not, and could not well be in the nature of the problem, any absolute data applicable alike to all cases met with in practice. The factors which determined the bearing values of soil were not yet established, and it was that and contingent uncertainty, and the comparatively cheap insurance against which in many cases it afforded, that warranted the use of an appropriate form of reinforcement.

Calculations on the assumption that a foundation was akin to a beam either freely supported or fixed, a continuous girder over *n* supports, or even a cantilever over weak patches, were more or less empirical for the reason that such assumptions were incapable of accurate proof. What was tolerably certain was that the binding together into one monolithic slab of the steel mesh with the aggregate and matrix of which reinforced concrete was composed, given a covering margin of strength, ensured within reasonable limits a structure that would withstand any or almost all those stresses, and if movement did occur it was *en masse* and of smaller degree and effect than would be the case without reinforcement. On those grounds alone he believed, where there was doubt as to the adequacy of the bearing strength of a road bed, or as to the absence of weak places in such, suitable reinforcement was justifiable and probably beneficial.

BRIDGE CONSTRUCTION.

Messrs. J. F. Hawkins, M.Inst.C.E., and C. G. Mitchell, A.C.G.I., B.Sc., A.M.Inst. C.E., presented a joint paper entitled "The General Construction of Bridges," in which it was stated that for small span bridges up to 20 ft. the commonest types were :—

(*a*) Reinforced concrete slab ;
(*b*) Reinforced concrete beam and slab ;
(*c*) Rolled steel joists embedded in plain concrete ;
(*d*) Steel troughing filled with concrete ;
(*e*) Brick or masonry arches ; and
(*f*) Reinforced concrete arches.

Type (*a*) was particularly suitable for very small spans, but for spans over 12 to 15 ft. it was generally more economical to use type (*b*), as the thickness of slab required would be excessive. These types should require no maintenance.

Type (*c*), with joists spaced at about 2 ft. 6 in. centres and having no steel reinforcement in the slab between joists, lent itself very well to the building of small spans, and being practically "fool-proof" could be constructed almost entirely with unskilled labour, a very important point at the present time. To make the best use of the concrete with this type it should be carried 2 or 3 ins. above the top flanges of the joists, which added materially to the strength. It was essential that the joists should be protected, and for that purpose a layer of asphalt should be laid over the concrete. It was by no means uncommon in this type for the underside of the joists to be covered with concrete, attachment being assisted by strips of expanded metal or other steel mesh wrapped round the bottom flange. That practice was not advocated, as the percolation of water through small cracks was liable to cause rusting, which was not easily seen. It was better to leave the underside of joists exposed and to paint them periodically. The maintenance charges on this type were low.

Type (*d*) had the advantages that it could be opened for traffic earlier than when the preceding types were used and that no shuttering was required. The underside of the troughs, however, required to be painted and a waterproof layer was necessary as in type (*c*).

Types (*c*) and (*d*) were easily widened.

Type (*e*) was still largely used for culverts.

Type (*f*), a reinforced concrete arch, was popular where deep earth fills had to be carried. It was essential in this case to provide not only for the final load but loading during formation of the earth bank. Failures of this type had occurred, not due to bad design or faulty workmanship, but to lack of appreciation of the unequal loading that occurred, resulting in stresses for which no provision had been made.

For single spans or a number of spans of 20–50 ft. there were four principal types :—

(*a*) Reinforced concrete beam and slab construction ;

(*b*) Reinforced concrete arch ;

(*c*) Brick or masonry arch ;

(*d*) Built-up steel girders and jack arches.

Type (*a*) was undoubtedly the most popular type at the present time. Generally it was found economical to use beams at about 5 to 6 ft. centres, as if those limits were much exceeded the slab became unduly thick, materially increasing the dead load of the structure. Where a number of adjacent spans were to be constructed, the use of continuous beams greatly reduced the bending moments at the centre of the spans, with the consequent saving in weight and material. Attention was being directed to bending moments in columns and piers where continuous girders were employed, and although much remained to be done those stresses could be determined with sufficient accuracy. The tendency was towards thinner columns and piers with heavier reinforcement, as these set up less interference with the slope of the beams at the point of support.

The distribution of wheel loads on slabs had been much discussed, and the Ministry of Transport's requirement that the maximum spread of the load should be limited to the area bounded by 45° lines down to the tensile steel had been termed unduly conservative. French, and particularly American, practice allowed more latitude in this respect. As against this it could be contended that whereas when a bridge of this type was supporting only one vehicle the live load stresses were distributed over a number of main girders, yet there was not the same relief to the slab from the wheel load. Arguing from this it would follow that with a bridge of moderate width a single load very much higher than the designed load could safely be carried provided the slab was strong enough to bear the increased wheel load.

In the design of Type (*b*), reinforced concrete arch or aches, the elastic method had now superseded the static method of determining the bending moments to be provided for. Where conditions allowed, arches certainly were preferred by the layman, as they offered more scope for architectural treatment and appealed to the æsthetic sense. They could be built with much smaller rises than brick or masonry arches, avoiding excessive gradients on the approaches. The design of the abutments of this type was generally good, as the elastic method of calculation gave the resultant thrust with sufficient accuracy. Care should, however, be taken not to strike the centering before the concrete had reached a reasonable strength.

With Types (*a*) and (*b*) no maintenance should be necessary.

Type (*c*), the brick or masonry arch, was still constructed in large numbers.

Type (*d*), of built-up steel girders with the deck supported on jack arches backed with concrete, was still largely used in carrying roads over railways. The proportion of dead to live load was rather high, thus influencing the cost. There was also a certain amount of painting required for maintenance. A layer of asphalt over the concrete deck prevented the percolation of water causing rusting of the girders. The exposed steel work on the underside required periodical painting.

For long spans the beam and slab reinforced concrete bridge was not used, as the depth of the beams became excessive and cut into the allowable head room.

Three types were commonly used :—

(*a*) Reinforced concrete ribbed arches ;

(*b*) Steel trusses or lattice girders with cross girders ;

(*c*) Built-up steel girders with jack arches.

Type (*a*), reinforced concrete arches of long span, were usually designed in this country as hingeless, whilst Continental practice was to make them hinged for dead load and hingeless for live load. This was attained by making use of temporary hinges till after the centering had been struck and the bridge loaded with its full dead load, then concreting up the hinges, so making the arch hingeless for live load. The points usually chosen for hinges were at the springings and crown. This method had the advantage of allowing the dead load stresses to be determined more exactly and of controlling the position of the line of pressure for dead load. It also avoided what might be the disadvantage of incorporating hinges in the finished structure. Some Continental engineers went further than this by using permanent hinges. In that way the structure could be made rather lighter.

Type (*b*) had advantages for very long spans. On the Continent a large number of reinforced concrete trusses had been constructed, but in this country had not so far found a place, except for a few braced bowstring girder bridges. Reinforced concrete trusses were designed very much on the lines of steel trusses, but were generally heavier. Their advantage lay in reducing maintenance to a minimum.

Where the spans were not too long Type (*c*), the built-up steel girder bridge with jack arches, was still used over railways, but their weight was against them on long spans, and they required considerable construction depth. In type they corresponded to the reinforced concrete beam and slab bridge which was not suitable for long spans.

Structures in reinforced concrete always involved a considerable amount of anxiety during construction owing to the frailty of the human factor. The design might be perfect but entirely vitiated by wrongly-placed steel or poorly-proportioned and badly-mixed concrete. The mixing of the concrete was an art, and required most careful watching.

A number of tests were made at the building of the new Streatley and Goring bridges over the Thames, and the strength of the concrete was found to be affected in many different ways. Thus, the crushing strength at 28 days of a 1 : 2 : 4 concrete made with perfectly clean sand but containing 32·4 per cent. of particles passing a 50 by 50 mesh sieve was 1,810 lbs. per sq. in., while concrete made with the same sand with all but 10 per cent. of the particles below 50 by 50 mesh removed gave 3,670 lbs. per sq. in. at the same period. One day some of the concrete was observed to be too wet and four cubes were made at the same time, in the same manner, and from the same batch. Two of these tested at 29 days gave 1,780 lbs. per sq. in. and at three months 3,370 lbs. per sq. in., showing the slow growth of strength with a wet concrete, an important feature when the bridge had to be loaded as quickly as possible. The results of these particular tests emphasised the need of careful grading of the material and of the measurement of the water.

THE EXHIBITION.

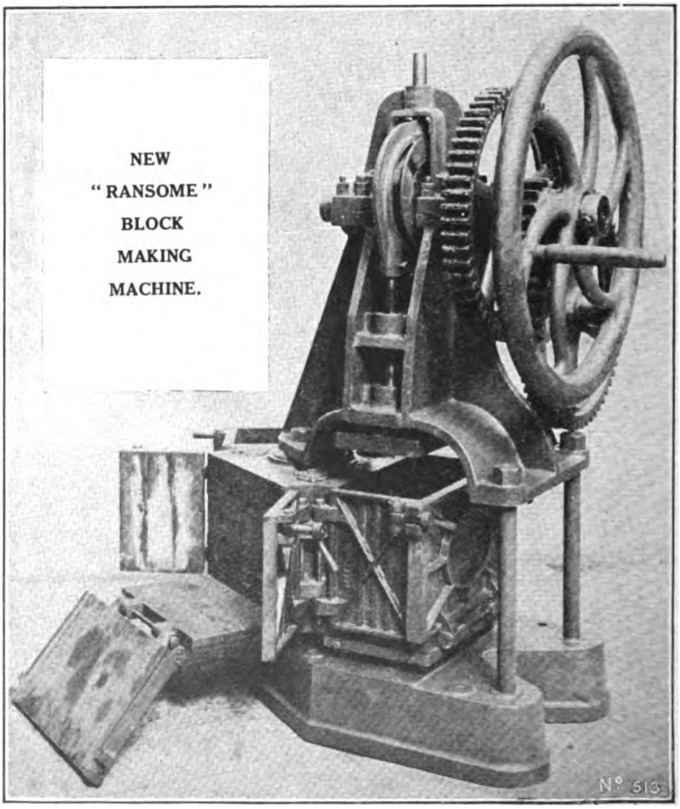
Whatever the magnitude of the road schemes which must inevitably be put in hand in the near future, it is evident from the exhibition held in conjunction with the congress that the means of carrying them out will not be lacking. The pick and shovel and rammer which were looked upon only a few years ago as the principal tools for road making have given way to pneumatic picks, steam shovels, mechanical mixe s, and steam rollers, and labour-saving and time-saving machines are now available for practically every operation in the construction of a modern road. Although the exhibition was chiefly devoted to materials and plant for road construction, there were several other exhibits of interest to the concrete industry from the constructional point of view.

The British Reinforced Concrete Engineering Co., Ltd. (1, Dickenson Street, Manchester), Messrs. Brown & Tawse, Ltd. (3, London Wall Buildings, E.C.), The Expanded Metal Co., Ltd. (York Mansion, Petty France, S.W.1), the Helical Bar and Engineering Co., Ltd. (72–4, Victoria Street, S.W.1), Messrs. Richard Hill & Co., Ltd. (Middlesbrough) and Messrs. Johnsons' Reinforced Concrete Engineering Co., Ltd. (24, Lever Street, Manchester), exhibited samples of their well-known reinforcement for road and building construction, while the

Walker-Weston Co., Ltd. (7, Wormwood Street, E.C.) also exhibited a film showing the use of their double-layer reinforcement in the construction of a road.

Among the numerous exhibits of machinery and plant, Messrs. John Fowler & Co. (Leeds), Ltd., of Leeds, showed road rollers and scarifiers and a concrete mixer complete with paraffin engine mounted on a road truck. " Rex " mixers and " Erie " shovels were exhibited by Messrs. Gaston, Ltd. (London Road, Acton Vale, W.3).

In addition to a good selection of their well-known concrete plant, The Ransome Machinery Co. (1920), Ltd., of 14–16, Grosvenor Gardens, S.W.1, exhibited a new type of block-making machine, the invention of Mr. Wm. Marriott, M.Inst.C.E. As will be seen from the illustration, the novel feature of this machine is the three revolving moulds, which enable a large number of blocks to be turned out per hour. While one mould is being filled the previously filled mould is being consolidated and the previously filled mould is being emptied. It is thus possible to keep the machine in continuous operation without waiting while a mould is being filled or emptied. Pressure is applied by hand through an eccentric gear which forces a wooden pressure block into the mould from the top. We are informed that the gearing gives a pressure of about 40 tons on the concrete in the mould.



Messrs. Stothert & Pitt, Ltd., of Orchard Street, S.W.1, exhibited their well-known " Victoria " concrete mixers. The latest pattern " Victoria " mixer has a capacity of 5 cu. ft., and is fitted with elevating loading hopper and water tank ; it is mounted on a strongly-built truck, which also carries the motive power in the form of a petrol engine.

A selection of their concrete plant, including mixers, block-making machines and elevators, was shown by Messrs. Winget, Ltd.

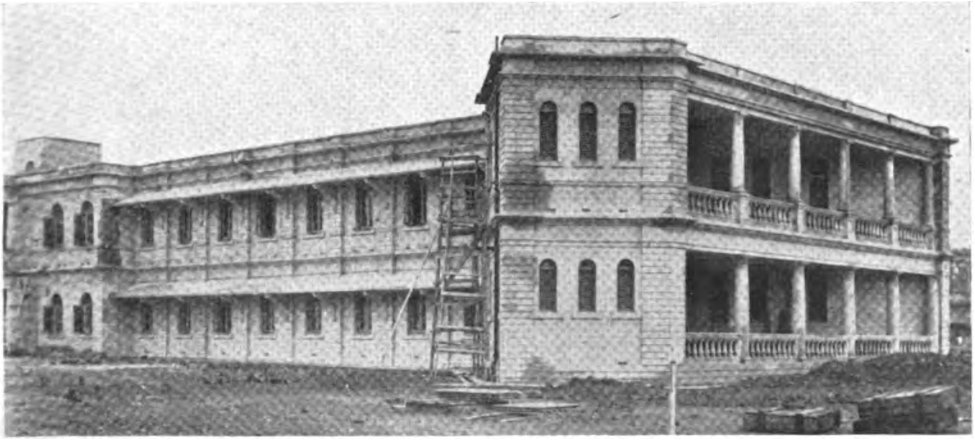
The " Holwin " system of concrete construction, shown by the Holwin Building Construction Co., Ltd., of Bush House, W.C.2, is a method by which the walls of a building are formed of pre-cast slabs without mortar joints. The slabs are made of the maximum length permitted by the design of the building and by considerations of handling, and of any desired section. The slabs are placed one above the other and fixed together by a steel rod being run through a cavity left in projecting strips of concrete on the top of the lower slab, and at the bottom of the upper slab, which form a male and female joint. Thus the slabs are interlaced with a steel rod throughout their entire length. An interesting feature of this system is that all the sections, blocks, quoins, etc., which are hollow, are of "spun" concrete made by the centrifugal process which, it is claimed, ensures the maximum strength combined with a smooth face.

The series of well-known machines supplied by Messrs. Millars' Timber and Trading Co., Ltd., of Pinners' Hall, E.C.2, were shown in operation under their own power. Of special interest to those concerned with concrete construction were the "Jaeger" mixers, the strongly-constructed plant which is so popular for road construction. Special features of this mixer are the automatic loader and hoisting drum and the automatic tipping water tank; the latter is a specially useful adjunct in regulating the quantity of water to each batch, and thus making the consistency of the mix a matter of precision which may be predetermined.

Messrs. C. A. Peters, Ltd., of Stores Road, Derby, exhibited a selection of panels showing the range of colours and textures imparted to concrete surfaces by their colouring materials, and by the use and exposure of an aggregate with a pleasing appearance.

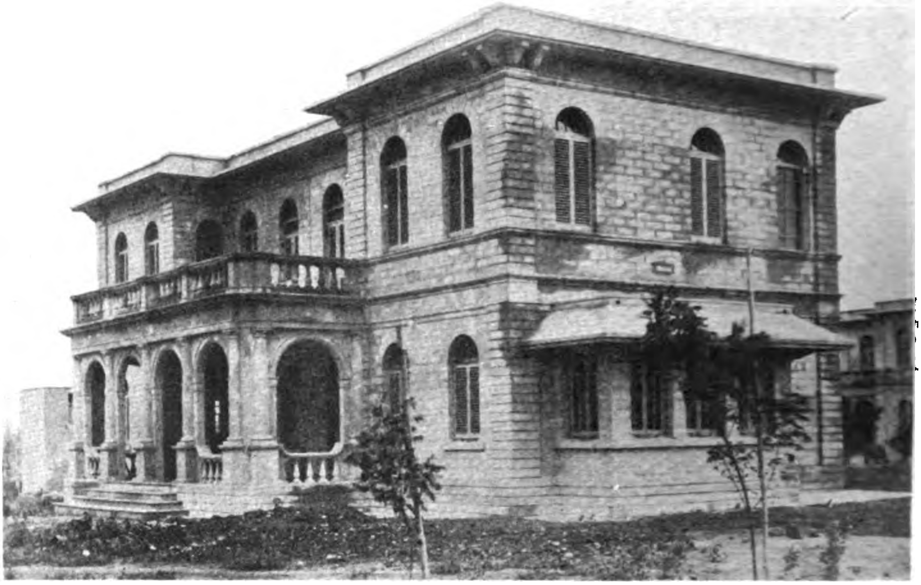
A very complete series of concrete products was shown by Messrs. Girling's Ferro-Concrete Co., Ltd., of Morley, Leeds, who are now supplying concrete posts for fencing along about 10 miles of Watling Street. Some sections of concrete posts were also shown to demonstrate the density which this firm obtains in the manufacture of its products.

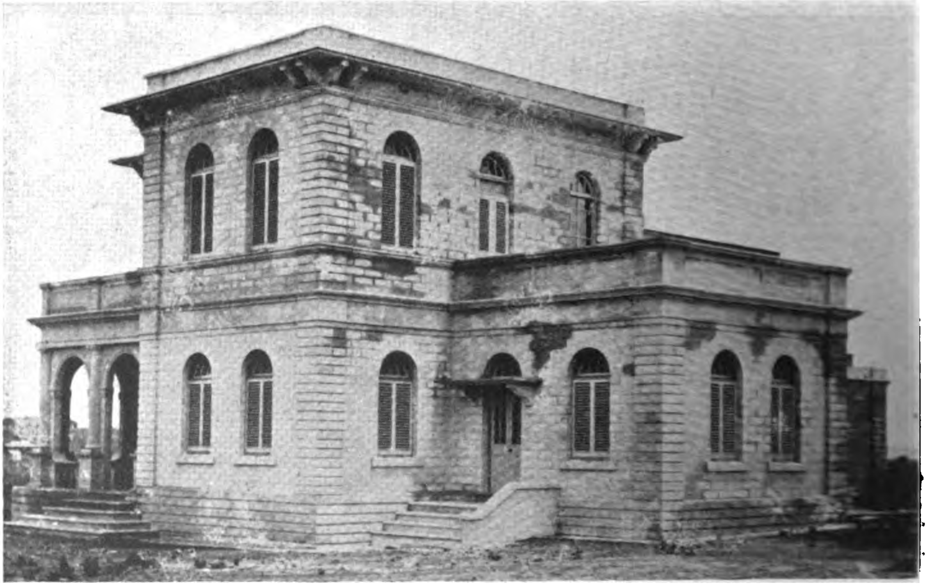
The application of silicate of soda for hardening concrete surfaces was demonstrated by Messrs. Brunner, Mond & Co., Ltd., of Northwich.



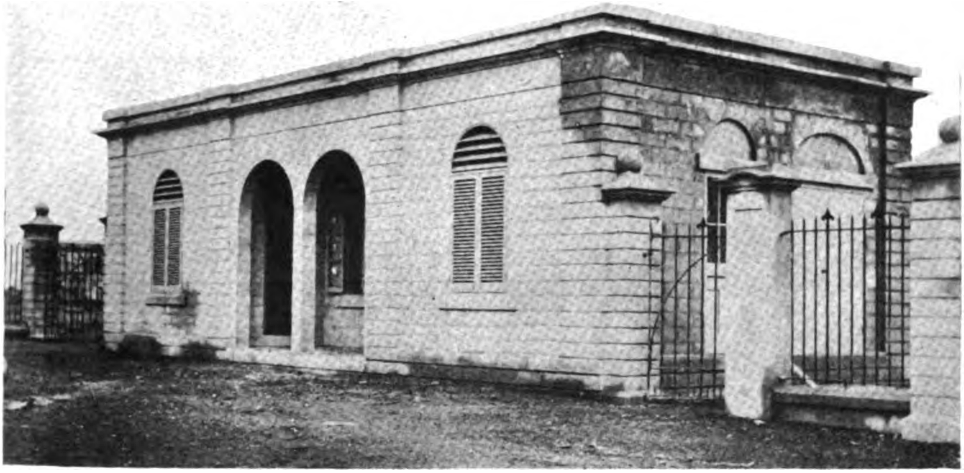
CONCRETE HOSPITAL IN SOUTH AFRICA.

This Hospital, situated at Accra, is, we are informed, the largest hospital in Africa. It is constructed entirely with concrete blocks made on "Cyclops" machines supplied by The Cyclops Concrete Co., of Liverpool. The contractors were Messrs. Thomas & Edge, of Woolwich. Other views of the hospital are given on the following pages. The house at Birkenhead, illustrated on p. 819, was also built of concrete blocks made on a "Cyclops" machine.

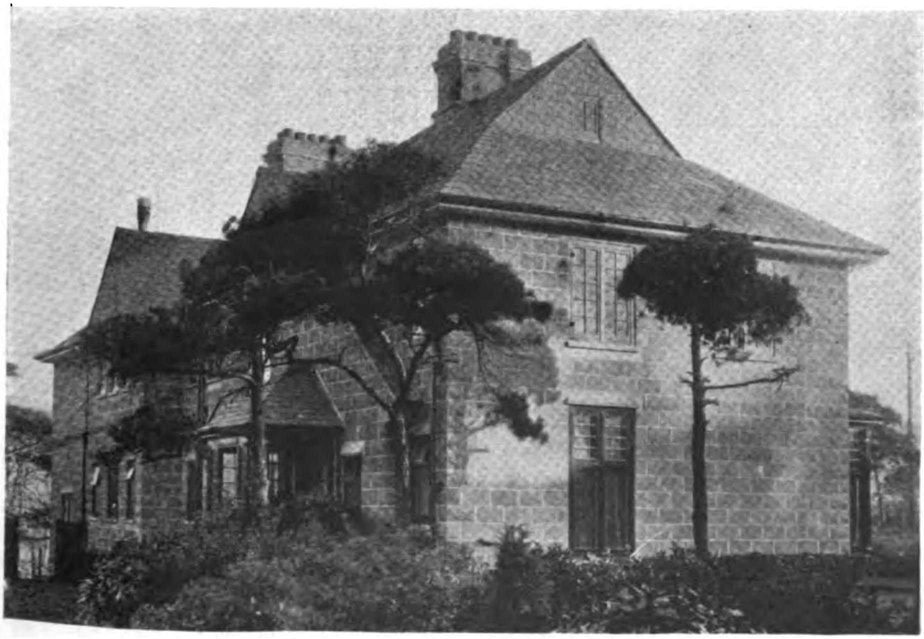




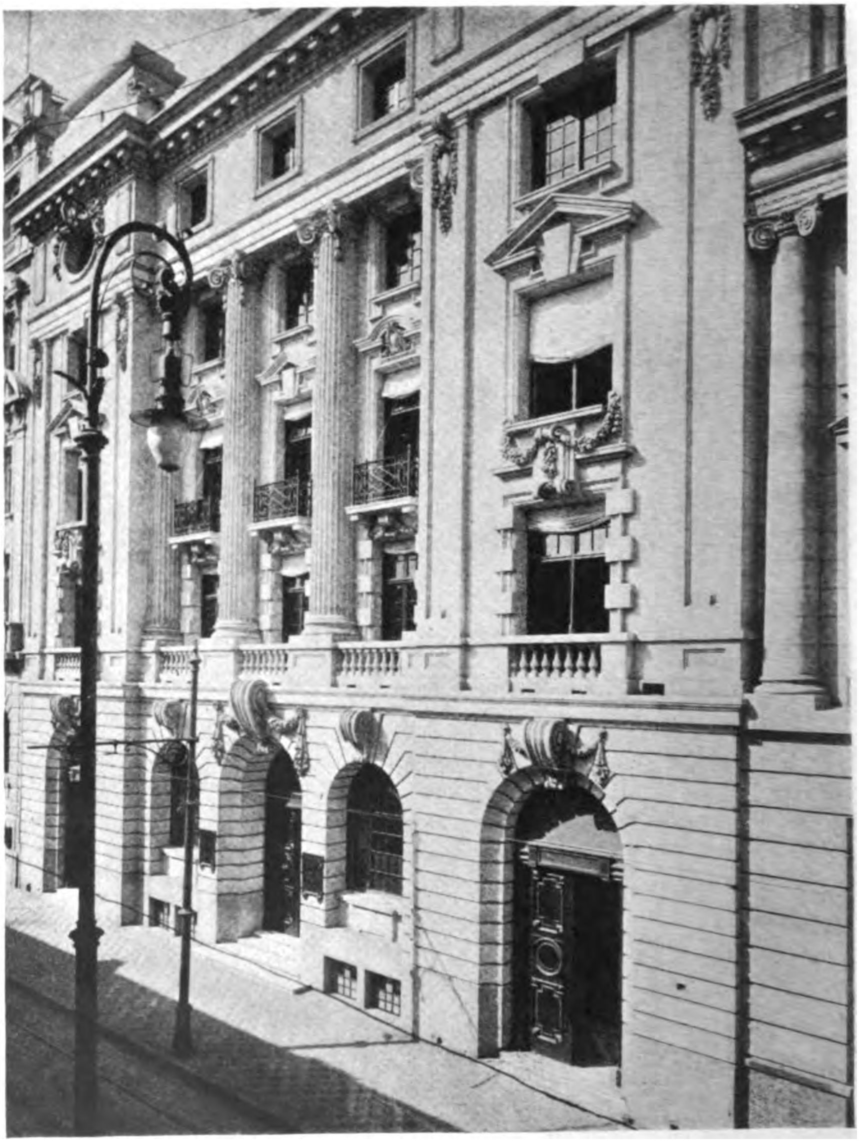
CONCRETE HOSPITAL IN SOUTH AFRICA. (*See p.* 817.)



PORTION OF A CONCRETE HOSPITAL IN SOUTH AFRICA. (*See p.* 817.)



CONCRETE HOUSE AT BIRKENHEAD. (*See p.* 817.)



THE SCHWAGER BUILDING, VALPARAISO: CAST STONE FAÇADE. (*See p* 821.)

A LIGHT CONCRETE AGGREGATE.

A Continental contemporary states :—

" An exceptionally light concrete aggregate offers advantages where great strength is not required, and is especially important in connection with concrete ships and barges. Such aggregate may be regarded as solidified froths, and they are made most conveniently by blowing air into a molten or semi-molten mass in order to create a " froth " which solidifies on cooling. A. Frisak has recently investigated the limiting conditions for the production of good froths of high porosity (up to 97 per cent. of air-space) from molten slag, and found that such froths can only be produced in the presence of moisture ; dry air or superheated steam are useless. With a mixture of air and steam the porosity can be regulated within wide limits.

" Most commercial slags from blast furnaces can be made to yield good froths and sound aggregate, provided they do not contain more than 3 per cent. of ferrous oxide. All natural slags can be made into froths if they are sufficiently free from easily reducible oxides. The viscosity of the froth is higher than that of the slag from which it is made.

" As nearly all the pores are sealed, these froths are impermeable to water and can remain floating for an indefinite period. The manufacture of blocks from slag-froths was also investigated ; the most satisfactory method consists in passing the hot froth between a pair of water-cooled rolls, from which it passes on to an inclined water-cooled iron plate and then into a series of water-cooled moulds carried on an endless chain beneath the plate. The blocks so produced must be annealed and should be stored for some weeks prior to use. They are stronger than highly-porous blocks made of kieselguhr, have a similar thermal conductivity, but are impermeable. They are superior in heat-insulating power to other porous bricks."

THE SCHWAGER BUILDING, VALPARAISO.

This building (*see* p. 820) has recently been erected, by Messrs. Fredk. Sage & Co., Ltd. (of Valparaiso, and London), in Prat Street, Valparaiso. It is constructed of reinforced concrete throughout, the general construction being a combination of skeleton steel girder construction and round bar reinforcement in walls, piers, columns and beams. The concrete floor slabs and roof are reinforced with fabric reinforcements. The concrete floors are all lined with 1½ in. by 3 in. matchboard stuck to the concrete with hot pitch, and secret-nailed to 2 in. by 2 in. battens of dovetail form inlaid in the wet concrete. The office partitions of the upper floors are constructed of thin reinforced slabs of plaster of Paris.

The front of the building is finished in Portland cement toned to a cream colour, and chiselled *in situ* to represent dressed stone. Many of the enrichments are also carved *in situ*.

The basement, occupied by a restaurant, is finished throughout in white cement walls and columns.

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		s.	d.
Best Washed Sand	per yard	16	0
Clean Shingle, ¾ in. mesh	,,	14	0
,, ,, ¼ in. mesh	,,	15	0
Best British Portland Cement	per ton 58s. to	63	0
"Ferrocrete" Portland Cement	10s. per ton extra		

Boarding for Shuttering—

		Sawn. s.	*Sawn.* d.	*Wrot.* s.	*Wrot.* d.
1 in.	per square	23	6	27	6
1¼ in.	,,	29	6	33	6
1½ in.	,,	35	6	41	0

Sawn Timber for Struts and Supports—

3 in. by 4 in.	from £24 per standard
3 in. by 6 in. and 3 in. by 7 in.	,, £25 ,, ,,

Mild Steel Rods for Reinforcement—

		s.	d.
⅝ in. to 2⅞ in. Rounds	per cwt.	13	0
7/16 in. to ½ in. Rounds	,,	13	6
⅜ in. Rounds	,,	14	6
¼ in. Rounds	,,	15	6

MATERIAL AND LABOUR, INCLUDING 10 PER CENT. PROFIT.

(Based on Contracts up to £2,000.)

Portland Cement Concrete well mixed with Clean Water in proportion 1 : 2 : 4—

		s.	d.
Do. do. in foundation	per yard cube	45	0
Do. do. in columns	,, ,,	50	0
Do. do. in beams	,, ,,	50	0
Do. do. in floor slabs 4 in. thick	per yard super	5	3
Do. do. in floor slabs 6 in. thick	,, ,,	7	9
Do. do. in floor slabs 9 in. thick	,, ,,	11	3
Do. do. in walls 6 in. thick	,, ,,	8	0

(Add for hoisting 3s. 6d. per yard cube above ground-floor level.)

Steel Reinforcement, including Cutting, Bending and putting into Position and securing with Stout Binding Wire—

		s.	d.
From ¼ in. to ⅜ in.	per cwt.	23	6
,, 7/16 in. to ½ in.	,,	22	6
,, ⅝ in. to 2⅞ in.	,,	21	6

EXTRA LABOUR TO BENDS in ¼-in. rods, ½*d.*; ⅜-in. rods, 1*d.*; ½-in. rods, 1¼*d.*; ⅝-in. rods, 1½*d.*; ¾-in. rods, 1¾*d.*; ⅞-in. rods, 2*d.*; 1-in. rods, 2¼*d.*; 1⅛-in. rods, 3*d.*; 1¼-in. rods, 3½*d.*

EXTRA LABOUR TO HOOK BENDS: ¼ in., 1*d.*; ⅜ in., 2*d.*; ½ in., 2½*d.*; ⅝ in., 3*d.*; ¾ in., 3½*d.*; ⅞ in., 4*d.*; 1 in., 4½*d.*; 1⅛ in., 6*d.*; 1¼ in., 7*d.*

Shuttering—

		s.	d.
Shuttering and Supports for Concrete Walls (both sides measured)	per square	54	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	50	0
Do. do. in small quantities	per ft. super	0	9
Shuttering and Supports to Stanchions for easy removal, average 18 in. by 18 in.	per ft. super	0	10
Do. do. as last, in narrow widths	,, ,,	1	0
Do. do. to sides and soffits of beams average 9 in. by 12 in.	,, ,,	1	0
Do. do. as last, in narrow widths	,, ,,	1	2
Raking, cutting, and waste to shuttering	per ft. run	0	3
Labour, splay on ditto	,, ,,	0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	per ft. run	0	3

WAGES.—The rates of wages on which the above prices are based are those current in the London area, namely:—Carpenters and joiners, 1/7½ per hour; Labourers on building works, 1/2¾; Operators on concrete mixers and hoists, 1/3¾.

[* This Data is specially compiled for *Concrete and Constructional Engineering*, and is strictly copyright.]

MEMORANDA.

PROSPECTIVE NEW CONCRETE WORK.

Barnet.—*Road.*—A by-pass road is to be constructed from Barnet Hill to Hadley.

Baslow.—*Bridge.*—A scheme for a new bridge over the Derwent is being considered.

Birmingham.—*Reservoir.*—The construction of a reservoir at Frankley, with a storage capacity of 500,000,000 gallons, will be commenced next year.

Bradford.—*Roads.*—The M.T. has sanctioned grants towards the schemes for new roads from Harrogate Road to Idle, Queen's Road to Five Lanes End, Beckside Road to Clayton Road, and Stonehall Road to Harrogate Road.

Brislington. — *Houses.* — The Sutton Trustees propose to erect 193 houses at Brislington.

Bromsgrove.—*Houses.*—The R.D.C. will apply to the M.H. for sanction to build 100 houses.

Cardiganshire.—*Bridges.*—The C.C. is to make a grant towards the erection of bridges at Rhyd, Trebedw, and Henllan.

Castle Douglas.—*Service Tank.*—The T.C. has decided to construct a service tank with a capacity of 100,000 gallons, estimated to cost £2,900.

Coventry.—*Sewage Works.*—The Corporation has applied to the M.H. for sanction to borrow £45,000 in connection with the extension of the Coventry Sewage Works.

Cowes.—*Road.*—The U.D.C. proposes to construct a sea road to Gurnard, estimated to cost £20,000.

Darlington.—*Houses.*—The Streets Committee has approved of a scheme for the erection of 200 houses.

Darton.—*Reservoirs.*—The U.D.C. proposes to construct two reinforced concrete reservoirs, having each a capacity of 1,000,000 gallons.

Durham.—*Bridges.*—The C.C. has decided to proceed with the reconstruction of the county bridges at an estimated cost of £200,000.

Eastbourne. — *Groynes.* — The Corporation has received sanction from the M.H. to borrow £3,480 for the construction of groynes.

Edmonton.—*Sewage Works.*—The U.D.C. has a scheme on hand for remodelling the sewage works at an estimated cost of £100,000.

Frinton.—*Sea Wall.*—The U.D.C. will apply to the M.H. for sanction to borrow £10,000 for carrying out the completion of its sea-wall scheme.

Glamorgan. — *Road.* — The Glamorganshire Main Roads Committee in conjunction with the Monmouthshire C.C. has decided to apply to the M.T. for a grant for the construction of a new main road through the two counties.

Greenock.—*Road.*—A new concrete carriageway is to be constructed on the Esplanade, for a distance of over a mile.

Haverfordwest.—*Paving.*—The T.C. has applied for a grant for laying new concrete pavements at Cartlett.

Hove.—*Sea Wall.*—The Corporation will apply to the M.H. for sanction to borrow £28,000 for the extension of the sea wall and esplanade.

Lincoln.—*Concrete Houses.*—The T.C. has decided to erect sixteen houses of concrete, estimated to cost £490 per house.

Llanfrechfa. — *Reservoir.* — The U.D.C. proposes to build a new reservoir in conjunction with the Llantarnam U.D.C.

Neston.—*Reservoir.*—The U.D.C. proposes to construct a new reservoir.

Neston.—*Houses.*—The U.D.C. has adopted a scheme for the building of fifty cottages.

Northwood.—*Houses.*—The M.H. has approved of the scheme submitted by the Council for building fifty-one houses.

Nottingham.—*Concrete Retaining Wall.*—The T.C. proposes to proceed with the construction of concrete retaining walls.

Ossett.—*Houses.*—The T.C. has received sanction from the M.H. to borrow £10,424 for the erection of forty-seven houses.

Ossett.—*Road.*—The T.C. has received permission from the M.H. to borrow £15,118 for the construction of a new street from Station Road to Teall Street.

Penrith.—*Reservoir.*—The U.D.C. has adopted a scheme for the construction of a reservoir, estimated to cost £26,000.

Plymton.—*Reservoir.*—The R.D.C. has received sanction from the M.H. to borrow £7,000 to enlarge Elburton reservoir.

Port Talbot.—*Houses.*—The B.C. has a scheme for the erection of 400 houses.

Prestatyn.—*Promenade.*—The U.D.C. has a scheme for constructing a promenade along Bastion Road to the shore.

Rawreth.—*Bridge.*—The R.D.C. will apply to the M.T. for a grant towards the cost of constructing a reinforced concrete bridge at Rawreth Shot, estimated to cost £1,000.

Salford.—*Houses.*—The Corporation will apply to the M.H. for sanction to borrow £120,000 for the erection of 100 houses.

South Shields.—*Road.*—The T.C. is considering the construction of a new coast road from South Shields to Whitburn at a cost of £100,000.

Ventnor.—*Defence Work.*—The U.D.C. is considering the construction of new sea defences, estimated to cost £2,000.

Walsall.—*Houses.*—The T.C. has decided to erect fifty-one houses.

West Hartlepool. — *Promenade.* — The Corporation has applied to the M.H. for sanction to borrow £42,000 for the construction of a promenade 1,600 ft. in length.

Whitby.—*Sewage Works.*—The surveyor to the U.D.C. has been instructed to proceed with the Foreshore and Spa sewage disposal scheme.

Wrexham.—*Houses.*—The R.D.C. will apply to the M.H. for sanction to erect 200 houses.

TENDERS ACCEPTED.

Burton-on-Trent. — *Footbridge.* — The Corporation has accepted the tender of Messrs. G. Hodges & Son, Burton-on-Trent, at £690, for the construction of a concrete footbridge over the Silver Way branch of the river Trent.

Chapel-en-le-Frith.—*Sewage Disposal Works.*—The R.D.C. has accepted the tender of Messrs. Hayes & Son, Heaton Norris, at £8,048 1*s.* 6*d.*, for the construction of sewage disposal works at Bamford, Derbyshire.

Chesterfield.—*Reservoir.*—The Corporation has accepted the tender of Messrs. J. K. Ford & Weston, Ltd., Derby, at £3,397, for the construction of a reinforced concrete reservoir.

Dingwall.—*Pier.*—The Ross and Cromarty C.C. has accepted the tender of Messrs. Yorkshire Hennebique Contracting Co., Ltd., at £2,935 18*s.*, for the construction of the Badentarbat pier.

Ely.—*Reservoirs.*—The R.D.C. has accepted the tender of Messrs. H. Arnold & Son, Ltd., at £3,150, for the construction of two reinforced concrete water towers, each with a capacity of 100,000 gallons.

Hull.—*Concrete Work.*—The T.C. has accepted the tender of Messrs. Quibell & Son, at £19,587, for ferro-concrete work in connection with extension of water arrangements at the power house.

Hunsworth.—*Sewage Works.*—The U.D.C. has accepted the tender of Messrs. W. H. Jowett & Co., Cleckheaton, for sewage extensions to Brierley Marsh.

Loch Katrine.—The Glasgow T.C. has accepted the tender of Mr. John Smellie, at £25,341 19*s.* 2*d.*, for raising Loch Katrine Outlet Works and the inlet basins to tunnels at the upper end of the aqueducts at Loch Katrine.

Oadby.—*Sewage Works.*—The U.D.C. has accepted the tender of Messrs. Armstrong, Whitworth & Co., at £12,771 19*s.*, for the construction of sewage works.

Poole.—*Reservoir.*—The T.C. has provisionally accepted the tender of Messrs. G. Pollard & Co., Ltd., Taunton, at £31,079 6*s.* 4*d.*, for the construction of a reservoir.

Ramsgate.—*Sea Wall.*—The T.C. has accepted the tender of Messrs. Monnoyer British Construction Co., Ltd., at £12,500, for the construction of a sea wall on the foreshore at the St. Lawrence Cliff. (In place of Messrs. F. M. Willers & Co.'s tender, which was withdrawn.)

Rathdrum. — *Concrete Cottages.* — The R.D.C. has accepted the following tenders for the erection of concrete cottages: W. Kavanagh, (1) £374 4*s.*; P. Farrell, (2) £748; M. Turner, (1) £380; J. Dunne, (2) £800; W. J. Geohegan, (9) £3,473 2*s.*

Shepsed.—*Sewage Works.*—The U.D.C. has accepted the tender of Messrs. Brown & Barton, Shepsed, at £5,407 2*s.* 11*d.*, for works in connection with the extension of the sewage works at Shepsed.

Stromness.—*Pier.*—The tenders of Messrs. Saxild & Partners, Westminster, for the construction of a steel and concrete pier, at £18,000 and £7,266 4*s.* 9*d.*, have been accepted.

Tilbury.—*Roads.*—The U.D.C. has accepted the tender of Messrs. F. Mitchell & Son, Manchester, at £10,424 15*s.* 5*d.*, for the construction of reinforced concrete carriageways.

Tiverton.—*Open-air Bath.*—The T.C. has accepted the tender of Mr. W. H. Smith, Torquay, at £2,052 10*s.* 4*d.*, for the construction of a reinforced concrete open-air bath.

RECENT PATENT APPLICATIONS.

183,477.—G. M. Gouyaud: Method of constructing reinforced concrete arches without falsework or scaffoldings.

204,516.—J. E. Miller: Concrete bricks and blocks.

204,618.—A. H. T. Hamselin: Manufacture of concrete flags and paving-blocks.

205,172.—D. De Nagy: Jigging or shaking machines for moulds.

205,204.—E. Law: Concrete wall construction.

205,263.—J. A. Davidson: Concrete pipes.

205,338.—J. Christie and A. R. Lang.—Form for moulding concrete.

205,448.—Sabulite (Great Britain) Ltd., J. Bellingham and W. H. McCandlish: Machines for pressing and moulding concrete.

205,679.—J. W. Wood: Apparatus for moulding concrete flooring.

205,688.—H. E. Steinberg: Moulds for reinforced concrete moulding.

205,720.—W. P. Carpmael (Naamlooze Vennootschap Korrelbeton Maatschappij voor Woningboun in Poreuye Beton): Moulds for concrete.

NEW COMPANY REGISTERED.

BATEMAN'S CONCRETE WORKS (SLOUGH), LTD. (193,650). Registered November 9, Bush House, Aldwych, W.C.2. Manufacturers of all forms of pre-cast concrete. Nominal capital: £750 in 750 shares of £1 each. Directors to be appointed by subscribers; qualification, one share; remuneration to be voted. Subscribers: F. Bateman, 102 Victoria Street, S.W.1, H. A. Leaver, Bush House, Aldwych, W.C.

TRADE NOTICES.

In the operation of splitting and dressing slate, a large amount of spoil rock is produced, the proportion of finished slate to that quarried being usually about one in twenty. For this reason there are at the quarries enormous dumps of spoil rock for which little use has hitherto been found. During and since the war, however, geologists and scientists have been actively engaged on research work in connection with this material, which has led to the discovery of a very large variety of ways in which it may be utilised. One of the uses of this material is as a "filler," and it has now been placed on the market for this purpose, under the name "Fullersite," by Messrs. Fullersite, of 62, Baker Street, W.1. It is claimed that the use of this material in the manufacture of artificial stone and concrete products renders them hard, dense and strong. It is also recommended as a component for use in the manufacture of cement.

The Staveley Coal & Iron Co., Ltd., of Chesterfield, are now manufacturing cast-iron pipes lined with concrete.

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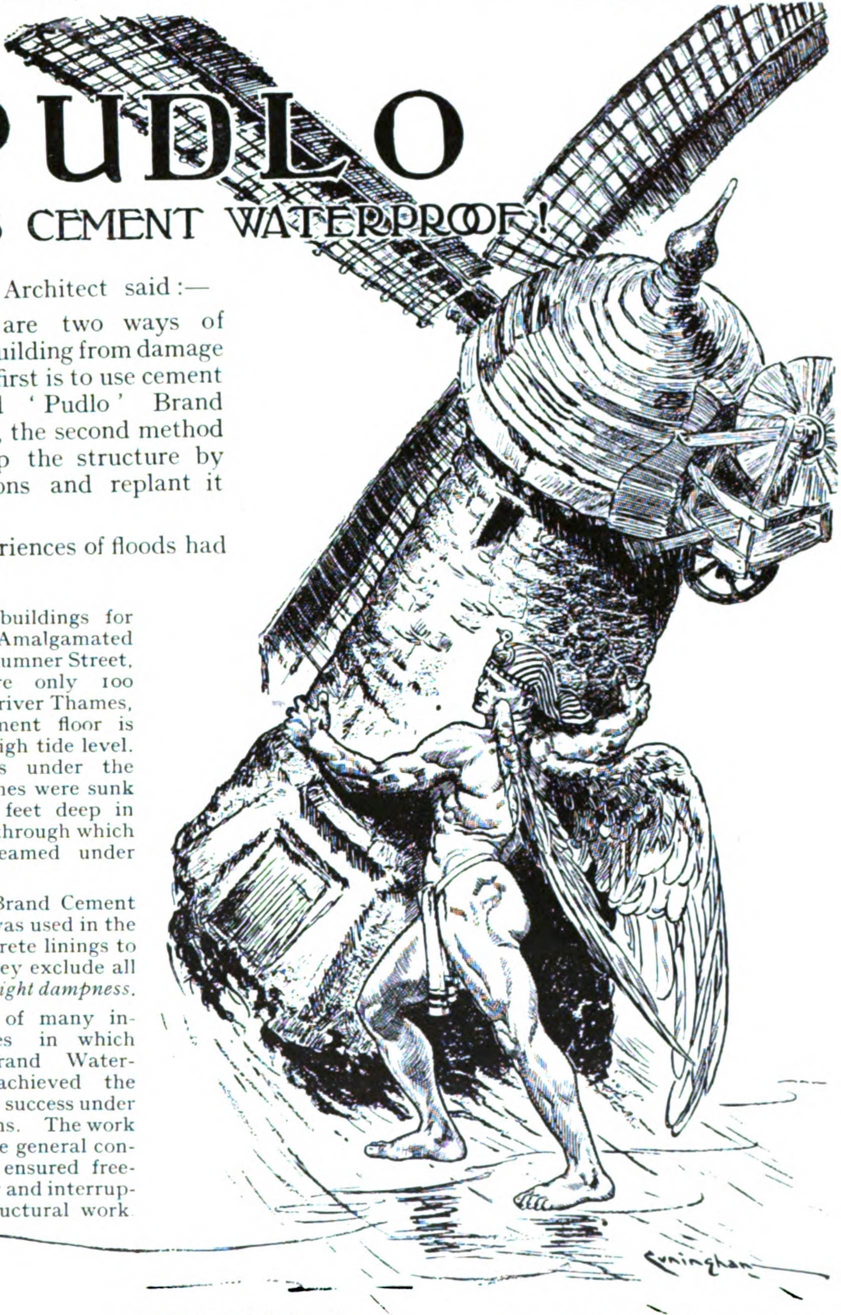

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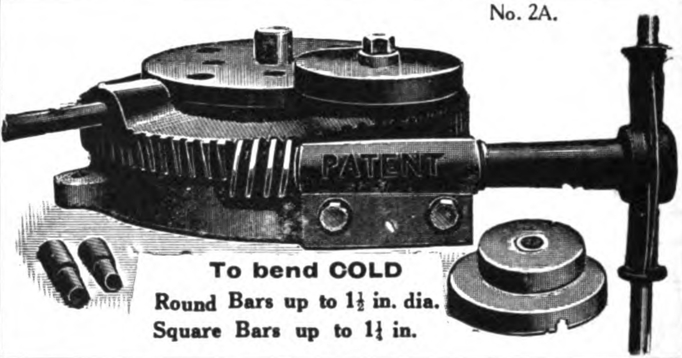

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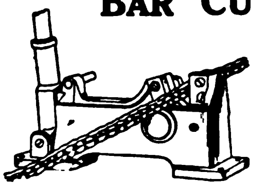

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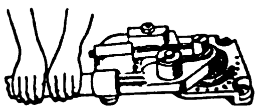

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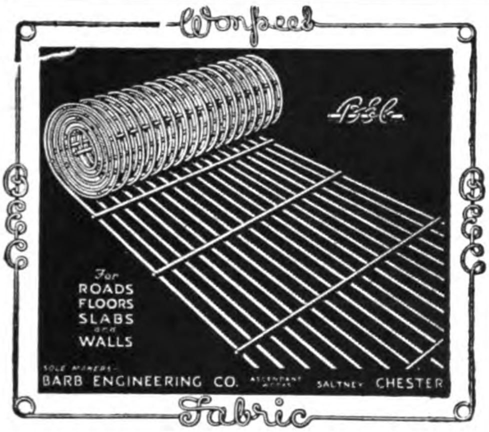
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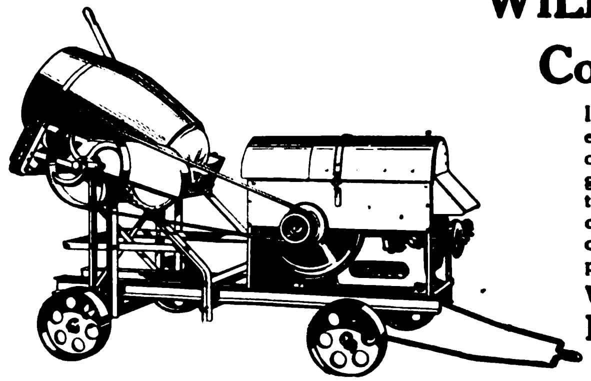

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